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"MODEL TESTING OF LOW-CRESTED BREAKWATERS"

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"MODEL TESTING OF LOW-CRESTED BREAKWATERS"

A Trident Scholar Project Report

by

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ABSTRACT

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Results indicate that wave transmission past rubble breakwaters can be approximated by solid breakwaters for deeply submerged breakwaters only. For higher breakwaters, wave transmission through governs so that rubble structures will differ from solid ones. Next, it is shown that as incident waves overtop reef breakwaters, they release their harmonics which in turn in propagate as linear waves behind the breakwater. For random waves, it is shown that wave transmission due to irregular waves can be modeled by regular waves of the same peak frequency and significant height. Based on the regular and irregular wave tests, a new parameter, $(F-Ru)/H_i$ is proposed to better represent transmission past a breakwater for all values of freeboard.

TABLE OF CONTENTS

NOMENCLATURE

A_1	spectral coefficients (cos coefficients)
A_2	spectral coefficients (cos coefficients)
a	empirical rough-slope runup coefficient
a_I	incident wave amplitude at a spectral line
a_R	reflected wave amplitude at a spectral line
B	breakwater crest width
B_1	spectral coefficients (sin coefficients)
B_2	spectral coefficients (sin coefficients)
b	empirical rough-slope runup coefficient
C	transmission by overtopping coefficient
C_1	empirical wave runup on smooth-slope coefficient
C_2	empirical wave runup on smooth-slope coefficient
C_3	empirical wave runup on smooth-slope coefficient
d_{50}	median stone diameter
F	breakwater freeboard
f	wave frequency
g	acceleration due to gravity
H_b	breaking wave height
H, H_i	incident wave height
H_r	reflected wave height
H_{rms}	root-mean-square wave height
H_s	significant wave height
H_t	transmitted wave height
K_r	reflection coefficient = H_r/H_i
K_t	transmission coefficient = H_t/H_i

NOMENCLATURE--CONTINUED

K_{to} wave transmission by overtopping coefficient

k wave number = $2\pi/L$

k_D stability coefficient

k_Δ layer coefficient

L wavelength

L_o deepwater wavelength = $gT^2/2\pi$

n number of stones used in cross section for determining B

Ru wave runup

S_r specific gravity of the rock used in the armor layer

T wave period

T_p period of peak energy density

W median weight of armor stone

w_r unit weight of the armor stone

$\Delta \ell$ gage spacing

η_{rms} root-mean-square water surface deflection

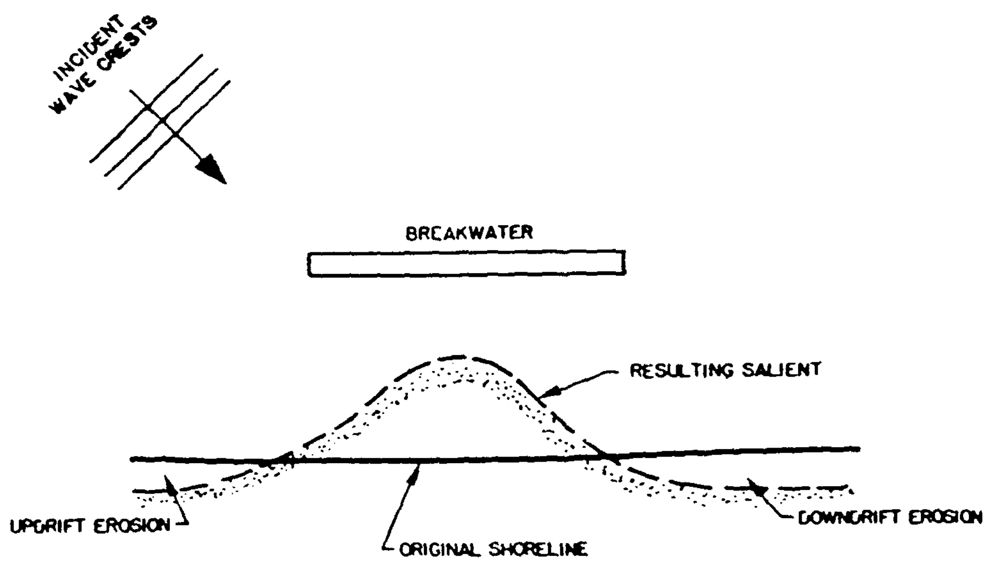
θ angle of face of breakwater

ξ surf similarity parameter

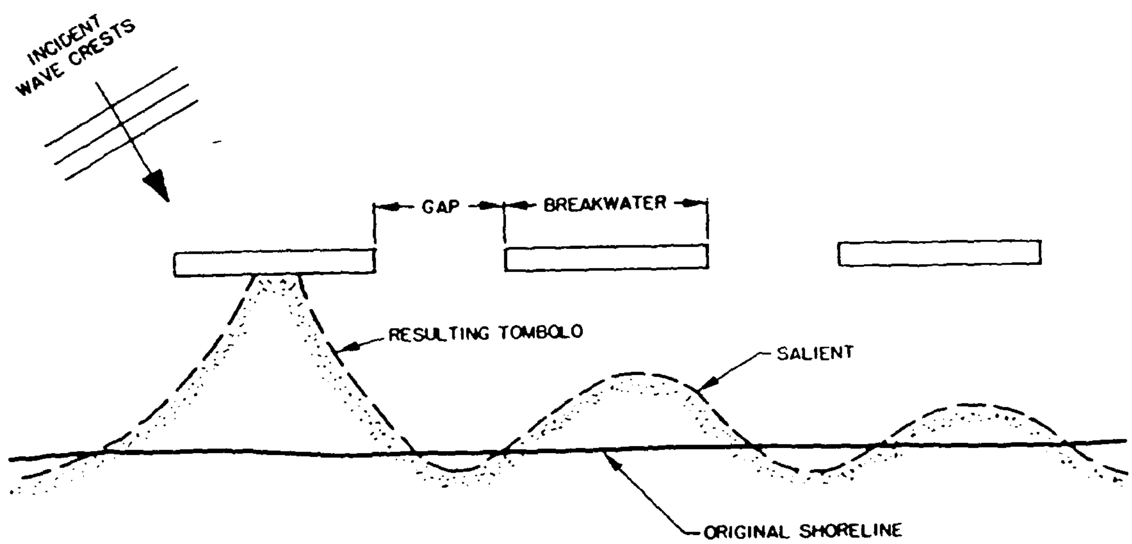
INTRODUCTION

Low-crested or reef breakwaters are rubble-mound structures which are built near the shoreline and which mimic natural reefs by reducing the wave energy that reaches the shoreline. The main purpose of a reef breakwater is to protect the shoreline or beach areas from erosion. Unlike traditional breakwaters used for harbor protection, reef breakwaters lack a multilayer design and are often made of a single stone size sufficient to resist wave attack. This, combined with their low crest height, makes these structures an attractive alternative for beach erosion control from a cost standpoint. As a result, reef breakwaters have recently become popular in the United States. In the past, they have been used heavily overseas, especially in the Europe and Japan, but they have lacked the public exposure in this country necessary for wide usage.

Reef breakwaters have been built as either a single structure to protect a very localized section of the beach, or as a segmented structure to provide larger area protection, as shown in Figure 1. These breakwaters have many advantages over more commonly accepted methods of beach erosion protection, such as bulkheads, groins, seawalls, and revetments. For example, reef breakwaters, if properly designed, can serve as effective protection against beach erosion while still allowing littoral drift or longshore



a. Single detached breakwater



b. Segmented detached breakwater (three segments)

Figure 1. Shoreline response behind reef breakwater from Dally and Pope (1986)

sediment transport to continue. In addition, because these breakwaters are built offshore they do not interfere with the existing layout of the beach, and are ideal for recreational beaches, unlike most other forms of shore protection. Furthermore, reef breakwaters create a sheltered area which makes them especially attractive as far as protection of coastal wetlands is concerned. This is an especially attractive feature in an area such as the Chesapeake Bay.

The main disadvantage associated with reef breakwaters is that they are more expensive to build than a land-based structure owing to the fact that they are constructed off-shore in the surf zone. For example, usually these structures are built in a region of breaking waves. They may also be built in an area where it may be too shallow for water-borne construction equipment, yet too deep for shore-based equipment. The advantages of these structures often far outweigh the disadvantages, however, as can be clearly seen by the rapid increase in the construction of these structures within the past decade, as summarized by Dally and Pope (1986).

At the present time, there is limited information available on the design of these relatively new structures. Unlike the rapid increase in the construction of these structures, the development of engineering design criteria for these structures has not followed quite the same rapid path. To date, the approach to designing these structures in the

United States has been very site specific in nature. Thus, either model tests, limited prototype tests, or in many cases both model and prototype tests, are utilized before the whole project is completed. At the present time, there are a limited number of numerical and physical modeling procedures which may be used, and most of these are rules-of-thumb rather than rigorous design criteria. This is what contributes to the very qualitative nature of much of the literature which is available on the subject of reef breakwaters today. Thus, it is still predominantly up to the engineer to analyze each site for wave climate and littoral transport regime and apply prior experience on the coast in question, in order to arrive at the best design for the area.

The present study is thus undertaken in order to investigate the wave transmission characteristics of reef breakwaters, or the breakwater's ability to dissipate incoming waves, under realistic conditions that would be encountered by an engineer in the field. The purpose of the study is to obtain quantitative data on wave transmission as a function of rubble-stone size, breakwater geometry, and incident wave conditions. Furthermore, it is the goal of this study to develop design criteria in a graphical or equation form which an engineer could apply in the design process of a reef breakwater.

This study is based on laboratory experiments in which small-scale models of reef breakwaters were subjected to varying incident wave conditions while transmitted waves were measured in the lee of the breakwater. The study is divided into four main parts: (1) regular wave study of solid (impermeable) breakwaters; (2) regular wave study of rubble (permeable) breakwaters; (3) irregular wave study of rubble breakwaters; and (4) harmonic analysis of transmitted regular waves.

The regular wave study of impermeable and permeable breakwaters was conducted to investigate the basic processes of wave transmission over and through the structure. The impermeable tests were conducted to investigate transmission by overtopping only. The rubble model tests, on the other hand, were conducted not only to investigate the transmission characteristics due to overtopping, but also those due to flow through the structure. In the end, the goal was to compare the solid and rubble tests and hopefully to discover those conditions in which transmission through the structure dominated, and those conditions in which transmission overtop of the structure dominated.

The irregular or random wave portion of the study was designed to compare transmission characteristics of rubble breakwaters under regular and irregular waves. Thus, the regular wave tests conducted in the first part of the study were compared to irregular wave tests conducted with breakwaters of similar geometries. The goal of this portion of the study was to determine whether or not regular waves were a good model or representation of statistically similar irregular waves.

The final portion of the study, the harmonic analysis portion, was conducted in order to test the hypothesis suggested by Professor Robert Dalrymple of The University of Delaware that transmitted waves break down into their harmonics as they pass over low-crested breakwaters. In this portion, transmitted waves were measured at one position in the lee of the breakwater and analyzed to determine harmonic components. These harmonics of the wave were then theoretically propagated to a point further behind the breakwater based on the assumption that the harmonics propagated as linear waves. At this "new" point, the waves were then reconstructed and compared to the actual wave history measured at that site by a wave gage.

BACKGROUND

The main purpose of a reef breakwater system is to dissipate the wave energy at a coastal location and thus prevent the erosion of the shoreline in the lee of the breakwater. The main gauge of how effective a breakwater is in dissipating the incoming wave energy is its transmission coefficient. The transmission coefficient is defined as the ratio of the wave height in the lee of the breakwater to the wave height of the incoming waves. As the transmission coefficient of a breakwater is altered, through different breakwater designs, various construction methods, or changes in oceanographic conditions, the effectiveness of the breakwater will change.

Reef breakwaters, as earlier stated, are ideal structures for use in a body of water such as the Chesapeake Bay. Indeed, at the present time, there are several examples of these structures either currently under construction or in place. One very good example of these structures is located in Annapolis, Maryland. As part of a shore protection program planned for the community of Bay Ridge, several reef breakwaters were proposed and are presently under construction, as shown in Figure 2. These structures will be subjected to very similar wave conditions as were tested in this study, and were built using the same design process as were the models in this study.

Figure 2. Reef breakwaters at Bay Ridge, Annapolis

SHORELINE RESPONSES

Reef breakwaters are designed to protect a certain section of the beach from direct wave attack, but depending on the wave transmission characteristics of the breakwaters, different shoreline responses may be invoked. Figure 1 shows the two different types of shoreline response associated with reef breakwaters. These are: (1) salients and (2) tombolos.

As illustrated, the salient is a sinusoidal beach response where the beach appears to approach the breakwater, but does not actually connect with the breakwater. The second type of response, the tombolo, looks much like the salient except that it does physically attach itself to the reef breakwater. As discussed by Dally and Pope (1986), the salient is generally accepted as the "proper" type of response, due to the fact that this type of response still allows the continuation of some longshore sediment transport and represents an adequate degree of wave dissipation. A tombolo, on the other hand, usually represents an over-dissipation of wave energy and is generally not as commonly seen as the salient.

EFFECT ON SHORELINE PROCESSES

There are a great many variables in reef breakwater design which the coastal engineer must consider, as shown in Figure 3. For example, the length, height, porosity, gap

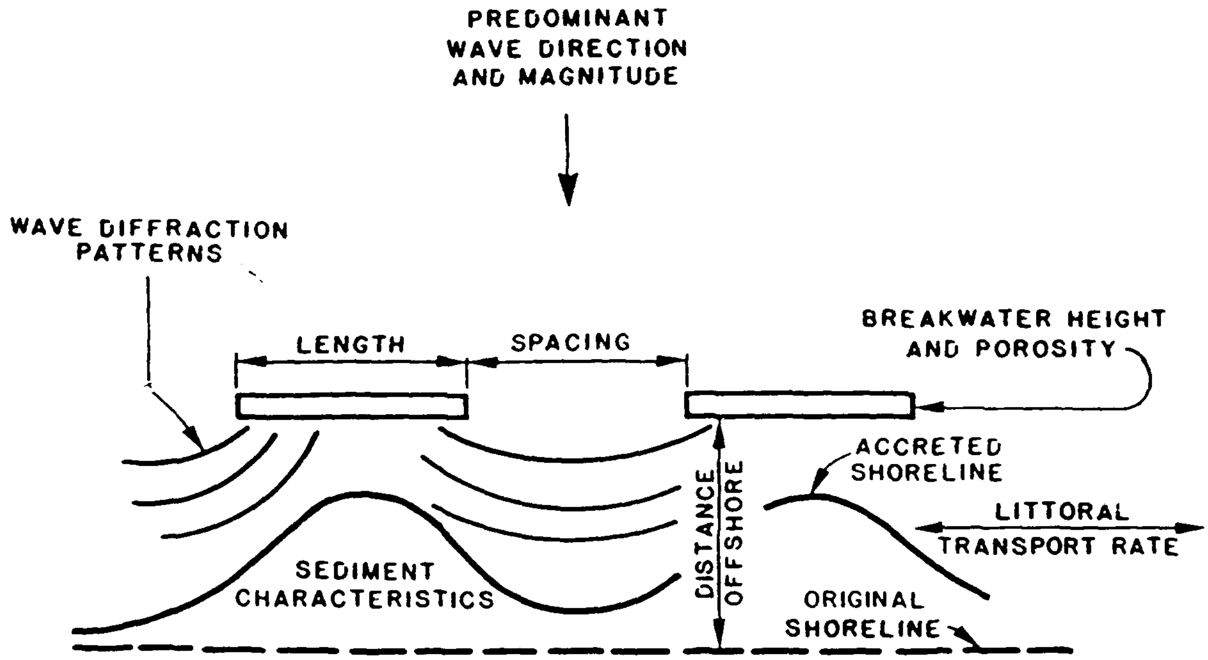


Figure 3. Reef breakwater variables from Dally and Pope (1986)

spacing, distance offshore, and orientation of the breakwater are all factors which may influence the effect of these structures on the surrounding coastal system, and serve to illustrate the complexity of the design problem. In addition, environmental factors such as the incident wave climate and prevailing littoral drift must also be considered. However, before describing the design considerations for an offshore breakwater, it is instructive to compare the reef breakwater to both a seawall and a groin, in order to study the effect that reef breakwaters have on coastal processes.

Until recently, groins and revetments or bulkheads have been the most popular forms of beach erosion control, in the United States. As shown in Figure 4, a groin is built perpendicular to the shoreline, whereas a revetment or bulkhead is built entirely on-land and parallel to the shore. A groin is purposely connected to the shoreline and does not appreciably reduce the incoming wave energy striking a beach. Revetments and bulkheads also do not reduce incoming wave energy and merely prevent erosion of the up-land region behind the structure. According to Dally and Pope (1986), groins are successful in trapping sand up to a point; however, following the accumulation of enough sediment, offshore sand losses will increase due to the fact that the sediment must travel out into deeper water around the groin. Revetments and bulkheads, on the other hand, are not effective in protecting a recreational beach. This is due to the fact that these

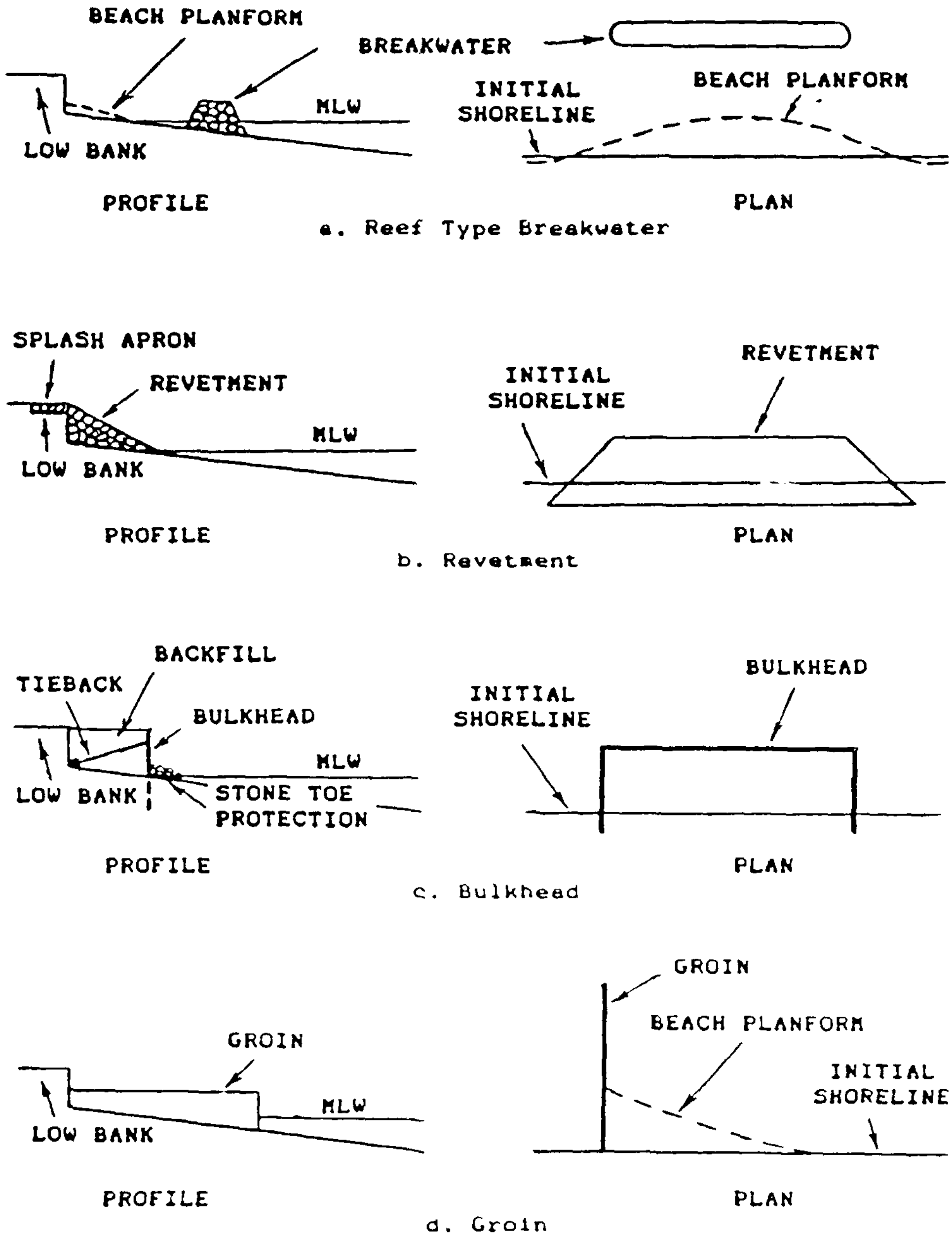


Figure 4. Coastal Protection Methods from Fulford (1984)

structures "sacrifice" the beach immediately in front of them in order to protect the up-land region; and, once the beach has significantly eroded in front of the structure, it may no longer be useful for recreational purposes.

Reef breakwaters, on the other hand, are built out in the surf zone and are predominately meant to reduce incoming wave energy. Detached breakwaters, if they are designed properly, do not completely stop the longshore sediment transport, except in the case where a tombolo is formed. In the case where salients are formed, there is still ample room for the near-shore transport to continue on the shore side of the reef, without increasing offshore losses. In addition, reef breakwaters usually enhance recreation by providing calm water and a sheltered beach suitable for swimming or many other activities. Ecologically, a reef breakwater has a great many advantages over other structures. For example, due to the fact that a reef breakwater is physically removed from the beach, it may provide new habitats for a variety of marine animals. In an area such as the Chesapeake Bay, where the bottom tends to be gently sloping and sandy, the voids found in the breakwater would provide animals with an additional habitat in which many would be able to find refuge from predators. In addition, reef breakwaters can be used in estuaries to protect wetlands or marsh regions from wave attack.

ENVIRONMENTAL DESIGN CONSIDERATIONS

There are a wide variety of environmental and oceanographic parameters that must be accounted for when designing offshore breakwaters. These include: (1) wave height; (2) wave period or frequency; (3) wave angle; (4) beach slope; (5) water level range; and (6) sediment size and supply. The first two of the above parameters are considered the most important, as they deal with the properties of the actual wave. Because of this, these were the primary environmental parameters which were varied in this study.

First, wave height controls beach response by obviously affecting the amount of energy that reaches the beach through or overtop of the breakwater. From linear wave theory, as given by McCormick (1973), it is shown that wave energy is proportional to wave height squared. Thus, by reducing wave height by a factor of two, for example, a reef breakwater then reduces the wave energy at the shoreline by a factor of four. Wave height is also responsible for determining the size of stones that are to be used in the breakwater design as will be shown.

Shoreline stability, however depends mainly on the longshore transport of sand, which is very sensitive to the wave height. For example, the CERC Formula, given in the Shore Protection Manual of the U.S. Army Corps of Engineers (1984) shows that sediment transport is proportional to wave height to the 5/2 power. Therefore, if a breakwater can reduce the incident wave height at a beach site by a factor of 2, then the longshore sediment transport will be reduced by a factor of 5.7, to only about 18% of the value without a breakwater. In addition, diffraction through the gaps of a segmented breakwater alters the wave angles striking the beach which has an effect on the longshore transport.

Wave period is an important parameter because it is a primary factor in determining the amount of energy that is actually transmitted into the shadow zone behind the breakwater. This is due to the fact that wave period determines the wavelength of the incident waves in any water depth. In general, the longer the wavelength, the more energy that finds itself transmitted through and around the ends of the breakwater. This effect on transmission by wavelength is one way in which tombolos may be inhibited. Finally, the combination of wavelength and wave height defines a third wave property, the wave steepness. The wave steepness is the wave height divided by the wavelength, and is also an important parameter in characterizing the transmission past a breakwater. Waves with low steepness have a nearly sinusoidal shape while waves with a high steepness are distorted with higher, more peaked crests and with flatter, broader troughs. High steepness waves are also closer to breaking than low steepness waves.

DESIGN CONSIDERATIONS FOR SHORELINE RESPONSE

Before any actual designs can be drawn of a breakwater itself, some preliminary decisions must be made concerning the amount of shoreline to be protected, the desired shoreline response, and the amount of actual beach area that is needed following the stabilization of the project. Once these decisions are made, it is up to the engineer to decide the design wave transmission coefficient of the breakwater, or the degree to which the incident wave energy must be attenuated by the breakwater in order for the desired end result to be reached. This is affected by: (1) the length of the breakwater; (2) the distance from the shoreline; (3) the permeability of the breakwater; (4) the freeboard of the breakwater; and (5) segmentation of the breakwater, i.e. the length of the gap spacing.

The length of the breakwater compared with its distance offshore is a primary indicator of whether a tombolo or a salient will be formed. If a salient is the desired planform, the U.S. Army Corps of Engineers Shore Protection Manual (1984) recommends that the length of the breakwater be one-half of the distance that the breakwater is located offshore from the original shoreline. This will ensure that the diffracted wave crests around the ends of the breakwater will intersect behind the breakwater before the undisturbed portions of the wave reach the shoreline. In the case of a tombolo, the Corps of Engineers recommends that the length be greater than one-half the distance offshore of the original shoreline. The larger the ratio of segment length to distance offshore, the larger the resulting tombolo, and beyond a point, there is even the possibility that a double tombolo may result.

There a three methods by which wave energy is transmitted past a reef breakwater: (1) through the breakwater; (2) overtop of the breakwater; and (3) around the ends of the breakwater or through the gaps in a segmented design. The first two methods of energy transfer are controlled by the permeability and freeboard of the structure, as illustrated in Figure 5. The wave transmission coefficient, K_t, is defined as the height of the transmitted wave divided by the height of the incident wave, and is shown to vary with breakwater height.

In the case of high breakwaters, transmission is limited to that which takes place through the permeable structure. This permeability is quite high in reef breakwaters due to the fact that they are usually built using only one size of rock, rather than in layers with a small-core stone as is found in large harbor breakwaters. Transmission through a breakwater is governed mainly by wave steepness. Low steepness waves are able to pass through a breakwater with very little energy dissipation, while high steepness waves dissipate more energy in travelling through the porous structure.

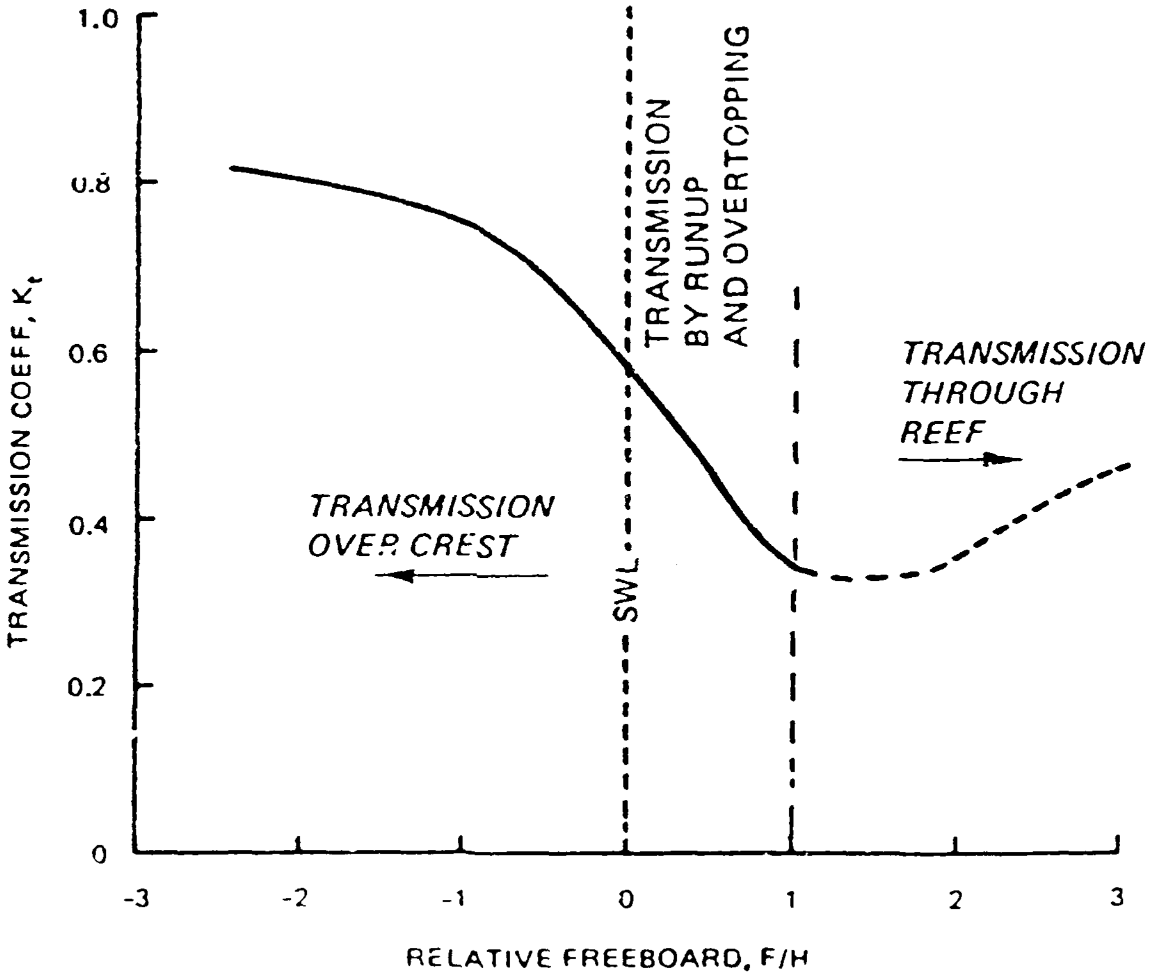


Figure 5. Modes of wave transmission through a breakwater from Ahrens (1987b)

In the case of low breakwaters, transmission is governed primarily by the freeboard of the breakwater. The freeboard of a reef breakwater is defined as the difference between the height of the breakwater and the depth of the water at the location of the structure. The freeboard of a reef breakwater is the greatest controlling factor influencing the amount of energy transmission past the structure by wave overtopping. Wave overtopping is the most variable of the three methods of energy transmission that were discussed above, and can be altered by the following factors: incident wave height, water level variation, incident wave period, freeboard of the structure, and crest width of the breakwater. According to Dally and Pope (1986), the advantage gained from varying the permeability, instead of the height of the breakwater, is that energy is transmitted through the structure at the incident wave frequency and is generally more predictable and regular than overtopping transmission. In the case of overtopping transmission, the wave will usually break as it overtops the structure, and in the process, "releases" several of its harmonic frequencies into the transmitted wave. These additional harmonics will affect the resulting beach planform and its stability in an unpredictable manner. Also, wave energy that would be transmitted through a structure is generally more uniform than diffracted wave energy, resulting in a more even shoreline.

PREVIOUS LABORATORY STUDIES

A great many of the reef breakwater projects that are built around the U.S. today are the result of site specific model tests in the laboratory. Due to the very complex nature of the problem there have been very few purely experimental studies done on these structures aimed at providing general design data. Goda (1969) tested vertical, solid structures for the effects of wave overtopping. Breakwater crests and heights were varied, along with various water level and wave conditions. Transmission and reflection coefficients were measured in the studies. A nonlinear empirical equation was proposed to define the transmission characteristics of these breakwaters as a function of relative freeboard, defined as the freeboard divided by incident wave height.

The most complete study to date of reef breakwaters was done by Seelig (1979 and 1980). A large number of two-dimensional tests were performed on a variety of cross sections. In total, 17 cross sections were tested, which varied anywhere from a solid breakwater to a more conventional design complete with concrete armor units. Transmission, reflection, and dissipation coefficients of these cross sections were measured under a variety of wave conditions and were plotted as a function of several different parameters, including wave steepness and freeboard. It was determined that transmission due to overtopping was, as with Goda (1969), strongly dependent upon the relative freeboard of the structures studied. In addition, transmission through the structure was found to be predicted reasonably well by a computer model written by Madsen and White (1976), but no simple design equation was proposed.

Ahrens (1987a, 1987b) performed both stability and transmission tests on low-crested breakwaters. The primary emphasis was on stability, and thus the tests were done with relatively small stone sizes that would move under wave attack. Crest heights, cross sections, and stone sizes, along with water conditions were varied. Several complex wave transmission equations were then proposed based on regression analyses. Ahrens (1987a) also proposed that a reef transmission variable, defined as the wave steepness times the breakwater bulk number, i.e., the average number of stones per average stone width in the cross section of the breakwater, was effective in predicting transmission through the structure.

In the present study, many of the same kinds of tests are performed, but with different ranges of conditions. The major difference between this study and that of Ahrens (1987a) is that the present work follows the actual design process more closely. As will be discussed, actual breakwaters are built with fairly large stone sizes that will not move under wave attack. In this study, these large stone sizes were tested such that the bulk number was an order of magnitude smaller than that used by Ahrens (1987a). Test conditions in this study are closer to those of Seelig (1980), however, some tests are conducted in shallower water and conventional reef breakwater cross-sections were investigated in more detail.

LABORATORY STUDY OF REEF BREAKWATERS

The purpose behind this study was to investigate transmission characteristics of low-crested or "reef" breakwaters under conditions similar to those that an engineer would face in the field. Thus, the problems faced were: (1) how to scale down a reef breakwater accurately; (2) how to model these structures in the laboratory accurately; and (3) how to design a "generic" model that would accurately model many different breakwater sites. The first step in the process was to scale down these structures accurately into a laboratory setting. Due to the fact that the models studied were generic in nature, and were not actual existing or planned structures, it was decided to use a preliminary scale of 1:15 based generally on breakwaters constructed in the Chesapeake Bay at Bay Ridge.

The next problem was how to represent accurately the complex wave transmission characteristics of reef breakwaters in the laboratory. In order to simplify the problem, it was decided to focus primarily on the transmission characteristics due to overtopping and transmission through the structure. Considering this, the solution was to study two-dimensional models of these structures, or in other words, a very small cross section of the structure. With a two-dimensional structure, the only transmission will be through or over the structure and not due to diffraction. Finally, all of the breakwater models tested were designed according to accepted and well-used design methods and practices, which will be discussed later.

WAVE BASIN SETUP

Laboratory tests were performed in the United States Naval Academy's Coastal Engineering Wave Basin. The wave basin measures 20.6 feet wide by 45 feet long at its widest and longest points respectively, as shown in Figure 6. Part of the tank was divided by 4 plexiglass walls in order to form three interior channels directly in front of the wavemaker. The widest channels measured 8 feet 2 inches in width and were located on the exterior portions of the basin. The interior channel measured 2 feet across and was used as the primary test channel for the two-dimensional transmission tests. The channel walls of the test section started 4 feet 4 inches from the wavemaker and extended 24 feet toward the back of the tank. The wavemaker consisted of a wave board which was moved back and forth in a piston-mode to generate waves. The wave board was capable of generating both regular and irregular waves. The test channel, as shown in Figures 7 and 8, consisted of two segments: one 8 feet long, and the other 16 feet long. Each segment measured two feet across. The first 8 foot section consisted of a 1:15 sloping beach, while the latter 16 feet comprised of a rigid, level false bottom as

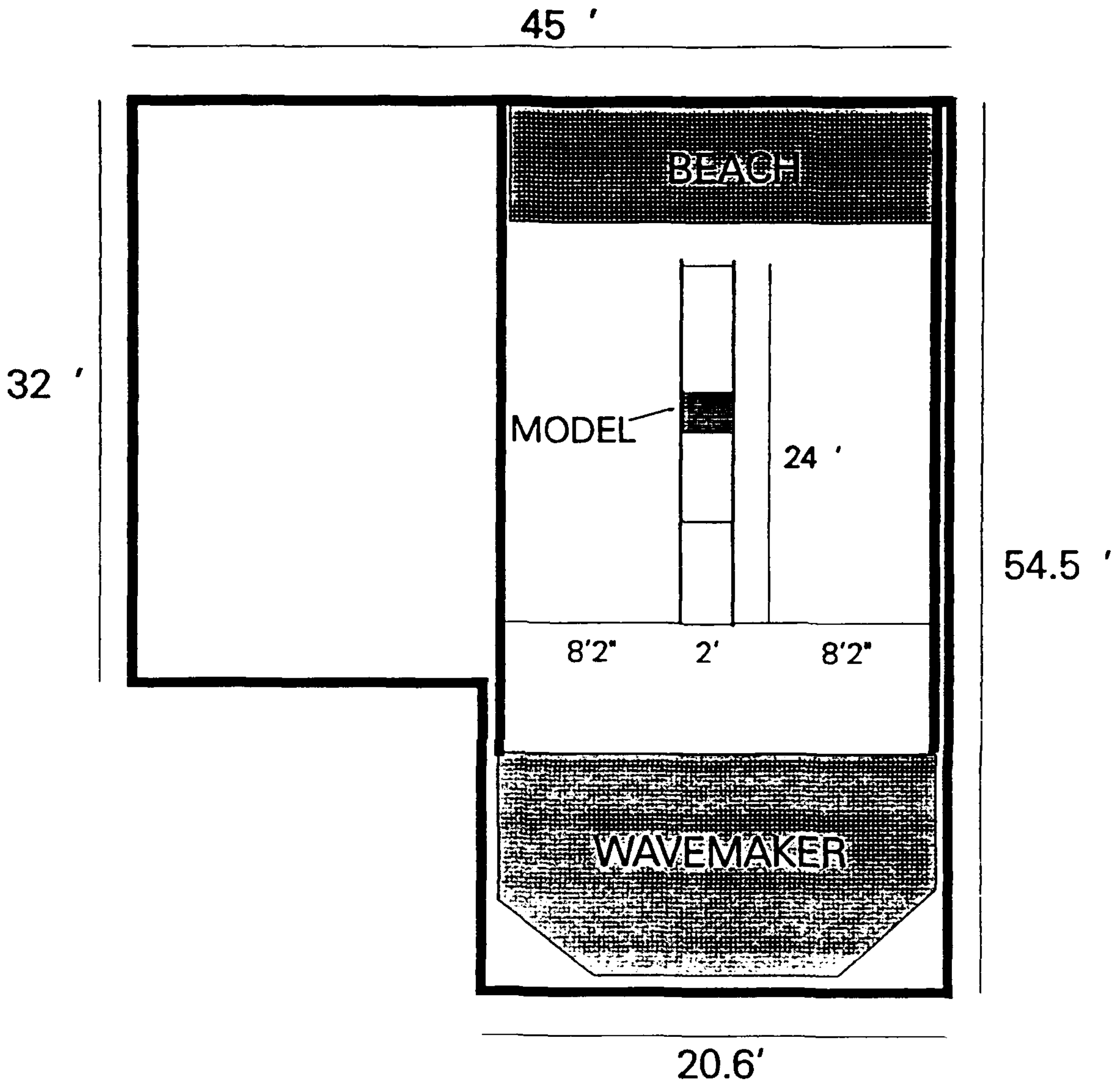


Figure 6. Top view - Coastal Wave Basin

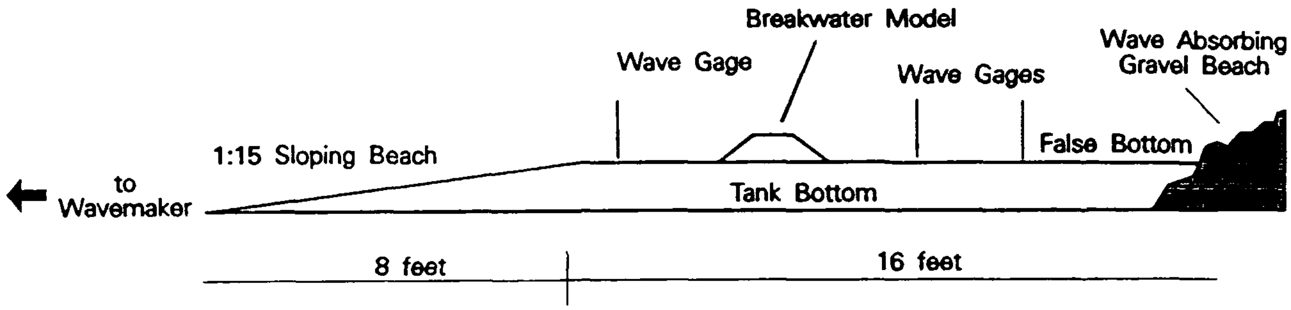


Figure 7. Side view of the test channel

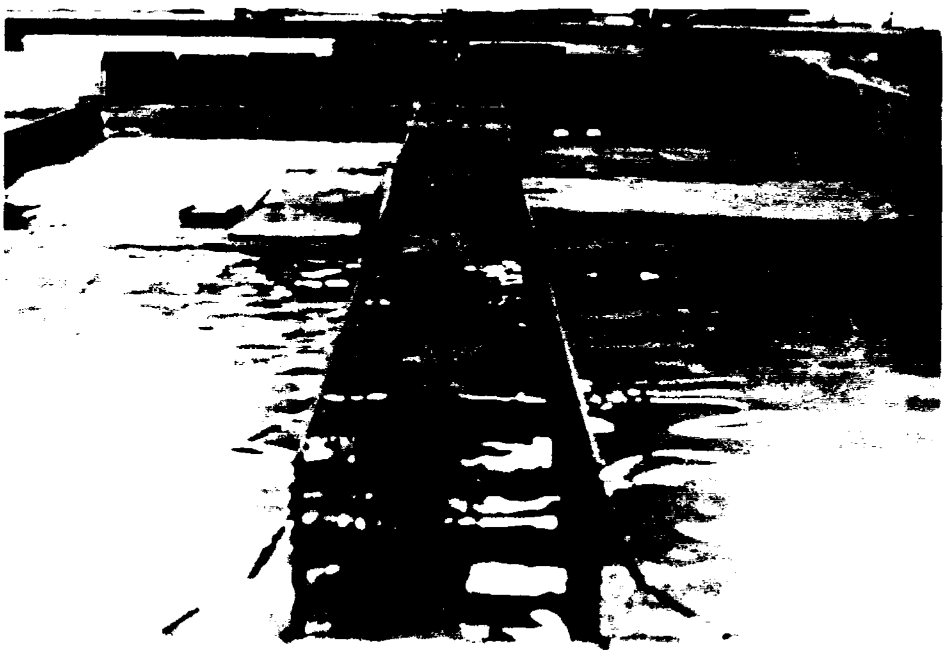


Figure 8. Test channel in operation

shown in Figure 7. The end of the test channel extended an extra 4 feet without the false bottom. A wave absorbing gravel beach was located in this final section. Between the end of the channel walls and the back wall of the wave tank, there was a 4.25 feet wide space that prevented the ponding of excess water behind the breakwater test sections. This area also served as space for a second wave absorbing beach, which was composed of 4 sloping layers of 2 inch thick horsehair packing material. This second beach, unlike the first, stretched across all three of the test channels. In addition, a swimming pool lane marker was used to attenuate the higher frequency waves in this region. Prior to testing of the breakwater sections, numerous tests were conducted to document the effectiveness of the wave absorption system. In all cases studied, the value of wave reflection did not exceed 10 percent.

WAVE GAGE SETUP

Capacitance wave gages were used to measure wave heights in front of and behind the breakwater during testing. The signals from these gages were run through an amplifier and signal conditioner and were finally sent to the data collection device. For the regular wave tests, three wave gages were generally used. Two gages were placed behind the breakwater and results were averaged to yield the height of the transmitted wave. The incident wave height was measured through the use of a single wave gage which was moved in front of the model. By moving the gage, the wave envelope caused by the constructive and destructive interference of the incident and reflected waves was measured. This was then analyzed in order to arrive at the values for incident and reflected wave height.

The irregular wave tests utilized 4 wave gages, two in front of the model and two in the lee of the structure. Both pairs of these gages constituted a Goda array, i.e., Goda and Suzuki (1976), which when analyzed has the capability of yielding the incident and reflected wave heights in both locations at all frequencies in the random sea. Finally, for the harmonic propagation tests, three wave gages were utilized. In this case, all three gages were located behind the model. These gages were placed at predetermined positions in order to test the hypothesis that waves break down into their various harmonics, which in turn propagate as linear waves, as they pass over or through the breakwater.

DATA COLLECTION METHODS

Two methods were used to collect data during the study. First, during the regular wave tests, a strip chart was used to record the wave heights both in front of and behind the breakwater. The second method of data collection involved the use of a personal computer-based data acquisition system. This system was used to collect data from the wave gages for a majority of the regular wave tests, and it was used exclusively for the irregular and prediction parts of the study. The data acquisition system provided a great deal of flexibility. It was capable of sampling the wave gages at different rates, and in addition, it also provided data analysis features, such as a Fast Fourier Transform routine.

BREAKWATER DESIGN

The breakwaters studied were two-dimensional models, of which the transmission characteristics of different wave conditions and breakwater freeboards were the most important parameters. As a result, several design variables were held fixed including: (1) the side slopes of the breakwater; (2) the weight of stone to be used in the breakwater; and (3) the crest width. Because this study attempted to test realistic scaled stone sizes and water conditions, the method of the selection of armor stone, crest width, and side slope geometry followed the same process that a design engineer would follow.

The first step of deciding upon the side slopes of the models was accomplished with the aid of an investigation into current reef breakwater projects. This study showed that most of these structures are built with a side slope of 1:1.5. This provides the breakwater with the steepest slope which is stable, and thus involves the least amount of material in the construction of the breakwater.

The second step in deciding upon breakwater stone size first required an investigation into the wave conditions present in the laboratory. In the case of this study, the highest waves which were capable of being generated in the laboratory were measured and found to be 4 inches. The specific weight of the limestone to be used in the building of the models was measured and found to be 166 lbs/ft^3. With these values, the next step is to select the size of the armor stone used in the breakwater.

The armor stones are those stones which are responsible for withstanding the incident wave energy, and a reef breakwater is traditionally constructed of a single mean stone size, with some variation about this mean size. The design of the armor layer is the most standard of any phase of the design process, yet it is also one of the most inexact. The equation which relates armor stone size to incoming wave height is the Hudson (1959) equation. This equation states that the weight of rock necessary to withstand a certain wave height is proportional to that height cubed, as:

$$W=\frac{w_r H^3}{K_D (S_r-1)^3 \cot\theta} \qquad \text{Eqn (1)}$$

where: W= weight of individual armor stone (lbs.)
w_r= unit weight of the armor stone (lb/ft^3)
H= incident wave height (ft.)
K_D= stability coefficient as discussed on pg. 37
S_r= specific gravity of the armor stone
θ= angle between the breakwater slope and the seafloor. (degrees)

Since the Hudson equation is the design standard in the field of coastal engineering for the design of breakwater armor stone, this equation was used in determining the weight, and thus the size of the armor stones, used in the breakwater models tested in the study. Entering equation (1) with the highest waves which were capable of being generated in the laboratory, the specific gravity and unit weight of the model stone, and the angle of the structure slope decided upon above, an armor stone weight of 0.4 pounds, with a diameter of approximately 2 inches was dictated. Following this analysis, the limestone was sieved and only rock that fell between the limits of 1.5 to 2.5 inches in diameter was accepted. In addition, each rock was weighed in order to make sure that it fell within the limits of 0.3 to 0.5 pounds.

The selection of a stable stone size may seem a very easy process, but there are several problems associated with Hudson's equation. These problems include: (1) its unknown stability coefficient K_D; and (2) its inability to account for wave period. First, the K_D coefficient attempts to account for all of the unknowns in the equation. According to the Shore Protection Manual of the U.S. Army Corps of Engineers (1984), the most important of these variables are:

(1) shape and interlocking friction between armor units
(2) thickness of the armor layer
(3) manner of placement of the armor units, i.e. precise vs. random placement
(4) surface roughness and edge sharpness
(5) type of incident wave, i.e. breaking or nonbreaking wave
(6) part of structure for which design is intended, either the trunk or the head
(7) angle of incidence of the wave attack
(8) model scale, if one is being used

Hudson (1959), in his model tests during the derivation of the formula, found that the average value of K_D for quarry rock, using different methods of placement, came out very close to the value of 1. With the advent of other model tests, currently recommended values for K_D for rubble breakwaters are 2.3, and this value was used in this study.

The second problem with the Hudson equation, the lack of dependence on wave period, is perhaps the most complex and troublesome to the design engineer. In the past, it had been observed that certain breakwaters were failing during storms were the maximum wave height was well below that for which the breakwater had been designed. When this situation was closely studied it was discovered that under certain wave period conditions, breakwaters were experiencing a resonance condition associated with the uprush and downrush of the attacking waves, as noted, for example, by Bruun (1989). A parameter, known as the surf similarity parameter, ξ, which is the ratio of structure slope, tan θ, to the square root of the wave steepness, H_i/L, seems to relate the stability of these structures to the incoming wave period the best. Bruun (1989) shows that when the surf similarity parameter is between 2 and 3, there seems to be the least stability of these structures.

The remaining aspects of the two-dimensional reef breakwater models were relatively easy to design following the design of the armor stone. The Shore Protection Manual of the U.S. Army Corps of Engineers (1984) dictates the following formula to use in designing the crest width of a breakwater:

$$B = n k_{\Delta} \left(\frac{W}{w_r} \right)^{\frac{1}{3}} \qquad \text{Eqn (2)}$$

where: B= crest width (ft.)
n= number of stones (3 is recommended minimum)
k_{Δ}= empirical layer coefficient
W= weight of armor units in cover layer (lb.)
w_r= unit weight of armor units (lb./ft^3)

The number of stones used for this model study was adopted as three, which is recommended by The Shore Protection Manual (1984), and would most likely be used by an engineer designing an actual reef breakwater. Based on equation (2), the value of crest width obtained was approximately 6 inches.

In conclusion, standard design procedure was followed. As a result, it was decided to test breakwaters with a median armor stone weight of 0.4 lbs, a crest width of 6 inches, side slopes of 1:1.5, and various positive and negative freeboards for various water depths. Thus, the breakwater was designed such that it would not be damaged during testing. However, the design methods are based on large traditional harbor breakwaters, and for reef breakwaters, it was not certain whether or not the armor stones would remain in place.

TEST CONDITIONS

Solid and rubble breakwaters were both tested during the course of this study. The solid models were constructed of PVC to the same geometry specifications as were decided for a rubble structure, i.e., a 1:1.5 side slope with a 6 inch crest width. The crest heights tested were also the same as for the rubble models, such that 4, 6, and 8 inch crest heights were tested in 4, 6, and 8 inches of water above the false bottom in the wave basin.

The solid breakwater tests were conducted using regular waves only. There were four frequencies tested at each crest height and water depth combination. These frequencies were 0.55, 0.7, 0.9, and 1.1 Hz. In turn, at each of these frequencies four wave heights were generated. These heights depended upon the water depth and the frequency, but extended to a height close to breaking. The regular wave tests for the rubble breakwater were run in exactly the same manner.

The irregular wave tests during the study were only conducted using rubble models. These tests utilized the same combinations of crest height and water depth as did the regular wave study. The difference between the regular and irregular wave tests was in the frequencies and wave heights tested. With the irregular waves, spectral peak frequencies of 0.7 and 0.9 Hz. were tested, and at each peak frequency, two significant heights were tested. Jonswap spectra were used in this study; an example is shown in Figure 9. Random seas are traditionally classified by these two parameters: (1) the peak period; and (2) the significant height. As can be seen in Figure 9, the spectrum concentrates a majority of its energy at one frequency, or in other words at its peak period. Second, because a random sea involves many different wave heights, the significant height is used to describe the sea. Significant height is a statistical parameter, and it can either be defined as 4 times the square root of the area under the spectrum or as the average of the highest 1/3 waves measured at a site.

The harmonic propagation tests, like the solid tests, only utilized regular waves, due to the nature of the hypothesis. For these tests, the same frequencies and wave heights were tested as in the previous regular wave tests. The only difference between these prediction tests and the regular wave tests with rubble breakwaters, aside from the different wave gage location, was that only breakwaters with zero or negative freeboard were tested in this section of the study.

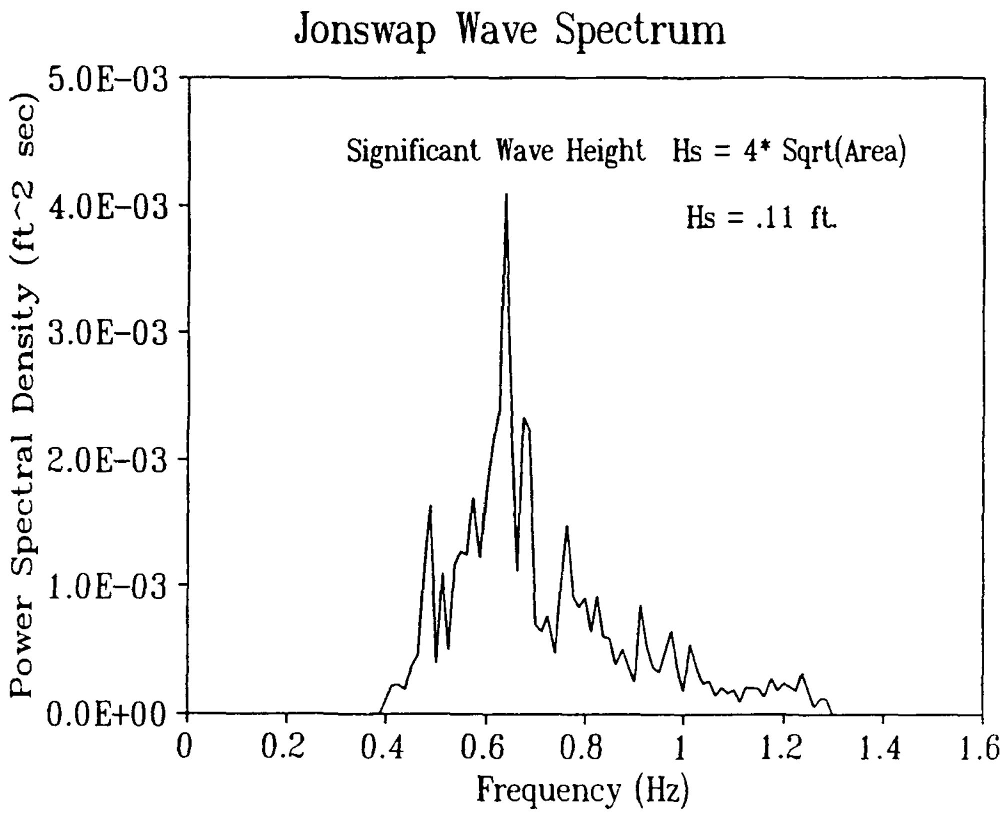


Figure 9. Irregular wave spectrum

DATA ANALYSIS

Following the setup of the laboratory and the design of the breakwater models, testing was initiated in the Coastal Engineering Wave Basin. The following test matrix was established for the regular and irregular wave portions of the study: (1) crest heights of 4 inches were tested in 4 and 6 inches of water above the false bottom; (2) crest heights of 6 inches were tested in 4, 6, and 8 inches of water above the false bottom; and (3) crest heights of 8 inches were tested in water depths of 6 and 8 inches above the false bottom. This matrix allowed positive and negative freeboard conditions of 2 inches to be investigated at three different water depths. In addition, the matrix also allowed the investigation of a zero freeboard condition at three different water depths.

Basic definitions associated with reef breakwater tests are shown in Figure 10. Breakwater freeboard is defined as the structure height, h_s, minus the water depth, h, or as $F = h_s - h$. Other important parameters discussed previously include the incident wave height, H_i, and wavelength, L, as well as the transmitted wave height, H_t. In some cases, wave runup, Ru, is also used as a primary variable. This is defined as the highest potential excursion of the water surface on the inclined slope of the breakwater.

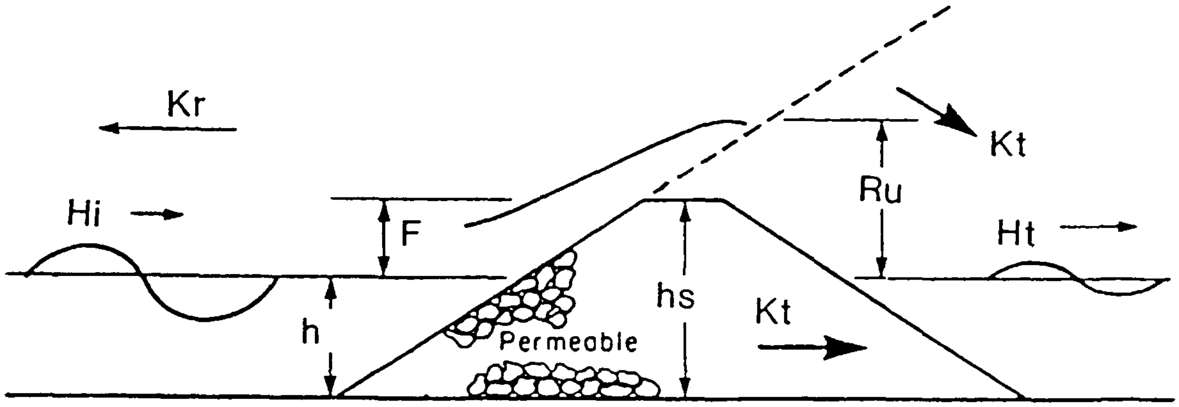


Figure 10. Reef breakwater definitions form Seelig (1980)

SOLID BREAKWATER--REGULAR WAVES

The first portion of the study was the regular wave study on solid or impermeable breakwater models as shown in Figure 11. The goal of this portion of the study was to investigate the transmission characteristics of reef breakwaters due to overtopping alone. For this portion, three wave gages were used to measure incident and transmitted wave heights, as shown in Figure 12. Two wave gages were placed 3 feet and 5 feet behind the breakwater and were used to measure the transmitted wave heights through averaging the output of the two gages. The third wave gage was located in front of the breakwater, between the structure and the wavemaker, and was used to measure the incident height of the waves striking the breakwater cross-section. This third gage was moved horizontally 6 feet in front of the breakwater, between the structure and the top of the sloping beach. By moving the gage, the envelope of the incident and reflected waves from the breakwater was measured, as described previously and illustrated in Figure 12. The method of data collection used in this portion of the study was the strip chart, from which wave heights could easily be determined.

Several different parameters were investigated in order to find the best way to represent the change in transmission coefficient values for different breakwater geometries and water conditions. The first parameter considered was also considered by Seelig (1980) and illustrates the change in K_t

Figure 11. Solid Breakwater cross-section

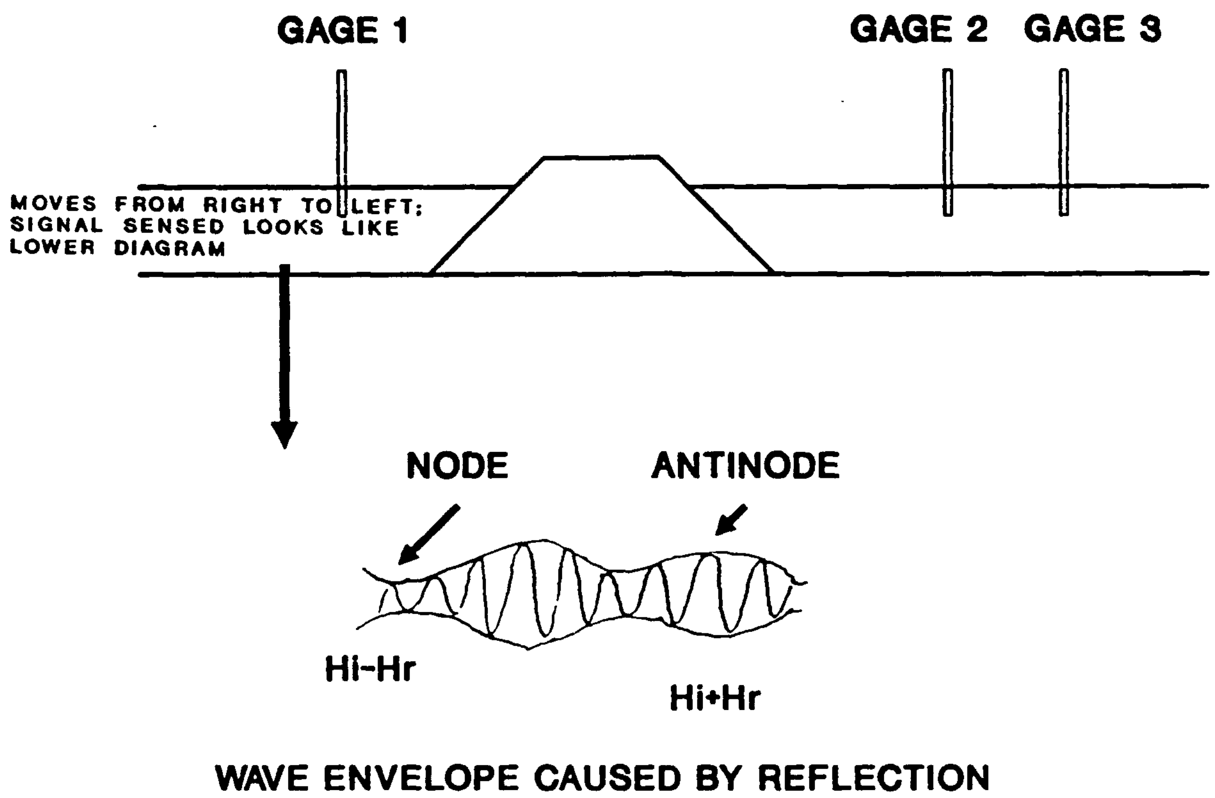


Figure 12. Wave gage setup for regular waves

values for various wave steepness conditions, H_i/gT^2, breakwater geometries, h/h_s, and relative water depths, h/gT^2. In these cases, gT^2 is related to the deep water wavelength, L_o, and is used as a characteristic length scale of the incident waves.

The dependence of wave transmission on wave steepness and relative water depth is shown in Figures 13-17. These figures do a very good job in illustrating the trends associated with various breakwater heights and wave steepness conditions. Breakwaters with a negative freeboard, ($h_s < h$), or a high water depth to structure height ratio, h/h_s greater than 1, as shown in Figures 13 and 14, exhibit a trend of very high transmission with very low steepness waves and of K_t values decreasing to about 0.5 for very high steepness waves. This can be attributed to the fact that the breakwater, being below the water level, allows waves of very low steepness to pass directly overhead with very little attenuation of the incident wave height. On the other hand, with very high steepness waves which are very close to breaking, the breakwater will succeed in "tripping" the wave. This "tripping" effect will cause the wave to dissipate a great deal of its energy on the breakwater, thus yielding a much lower K_t value of approximately 0.5.

The next breakwater to be considered is the very high breakwater with a positive freeboard. In Figures 15 and 16, these breakwaters with h/h_s less than 1 are shown. The trend

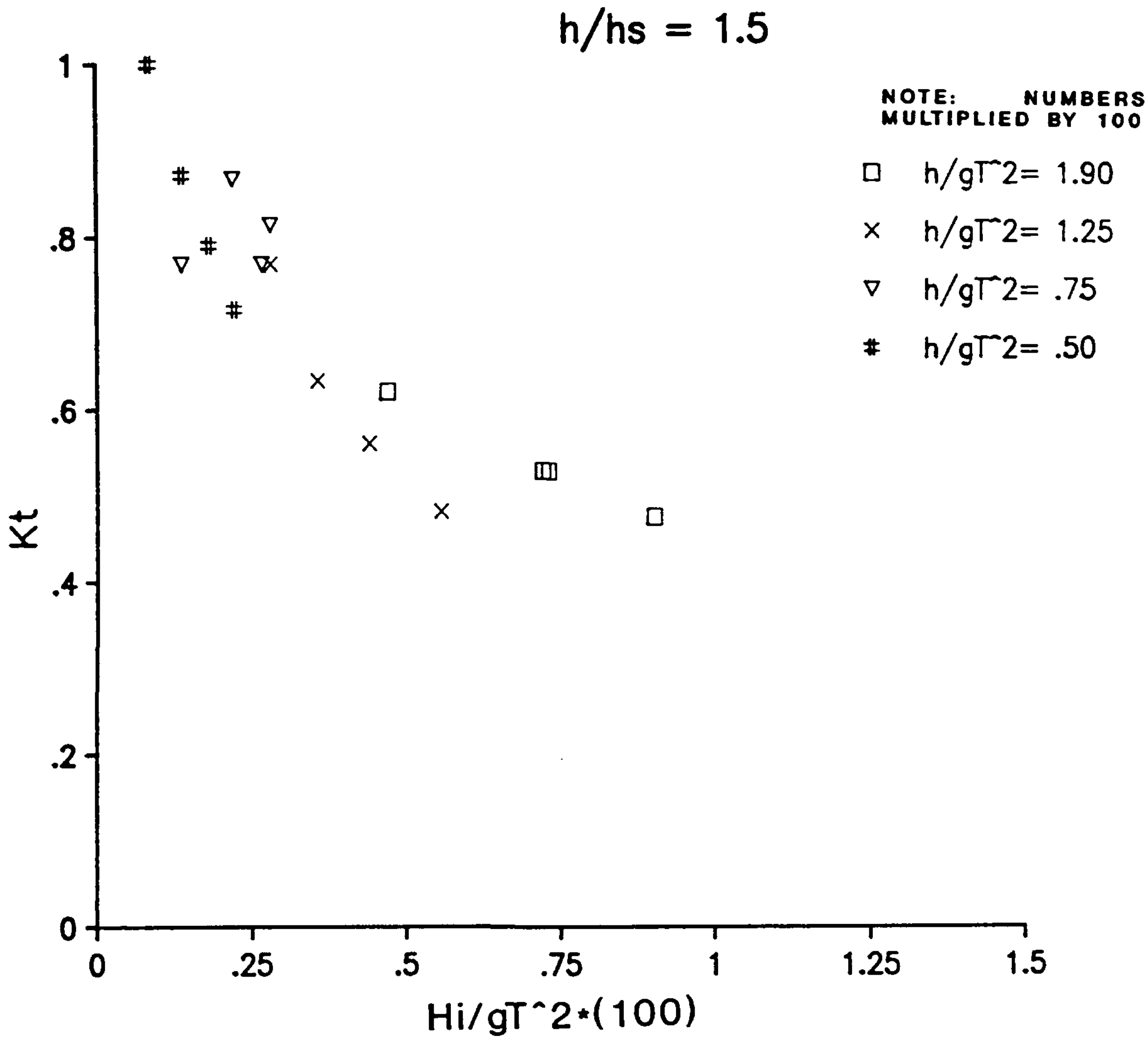


Figure 13. K_t versus wave steepness for solid breakwater

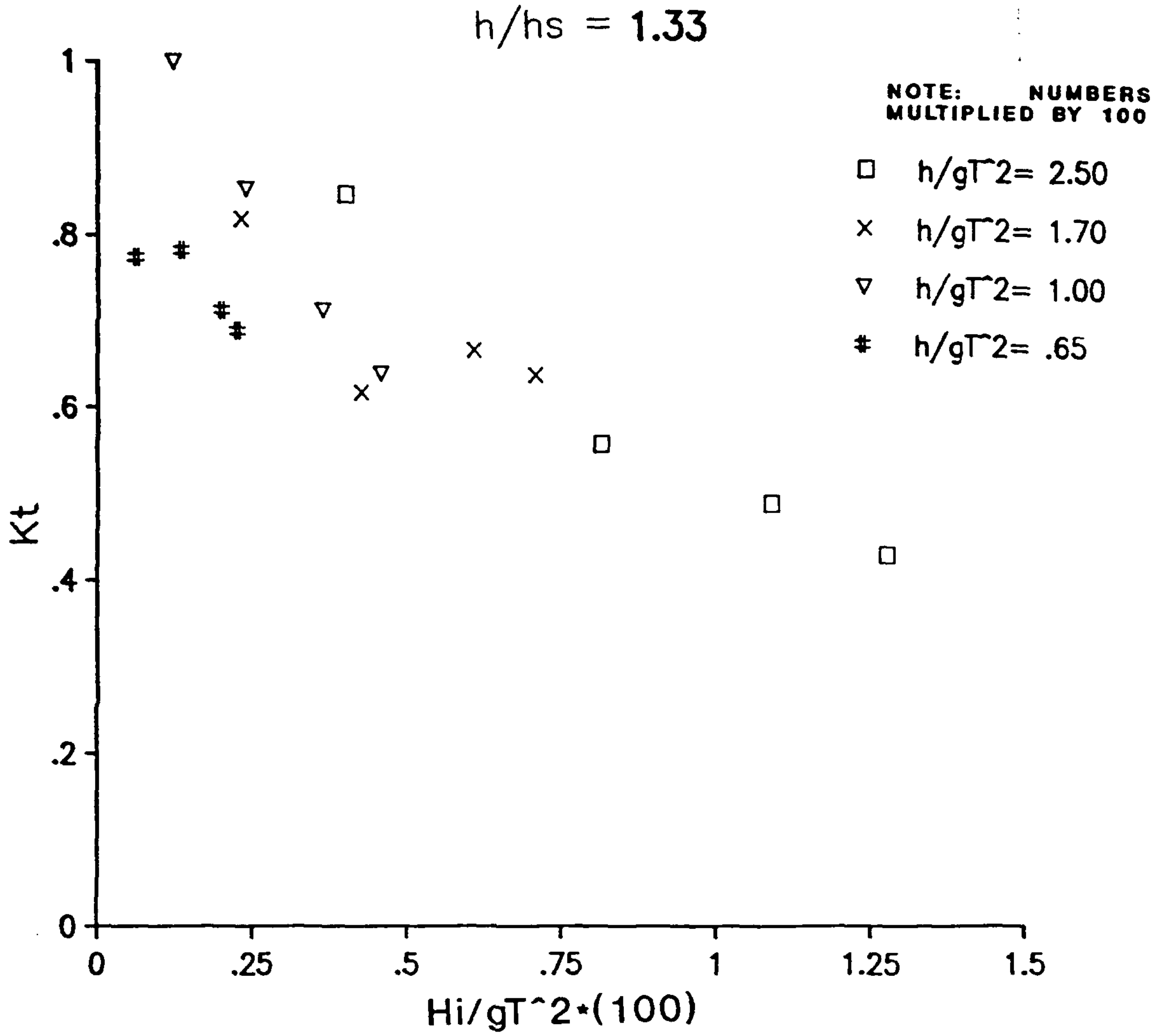


Figure 14. K_t versus wave steepness for solid breakwater

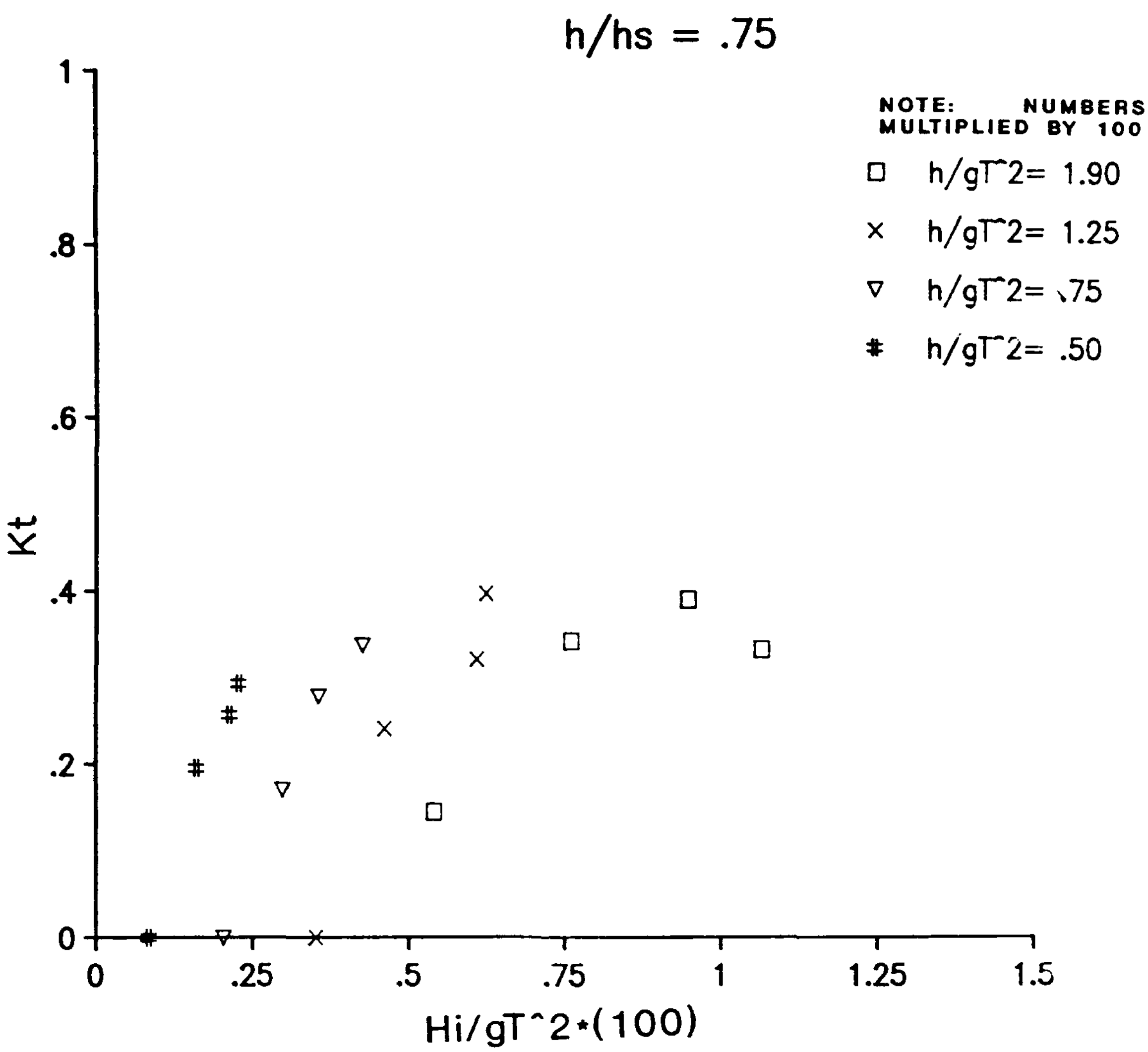


Figure 15. K_t versus wave steepness for solid breakwater

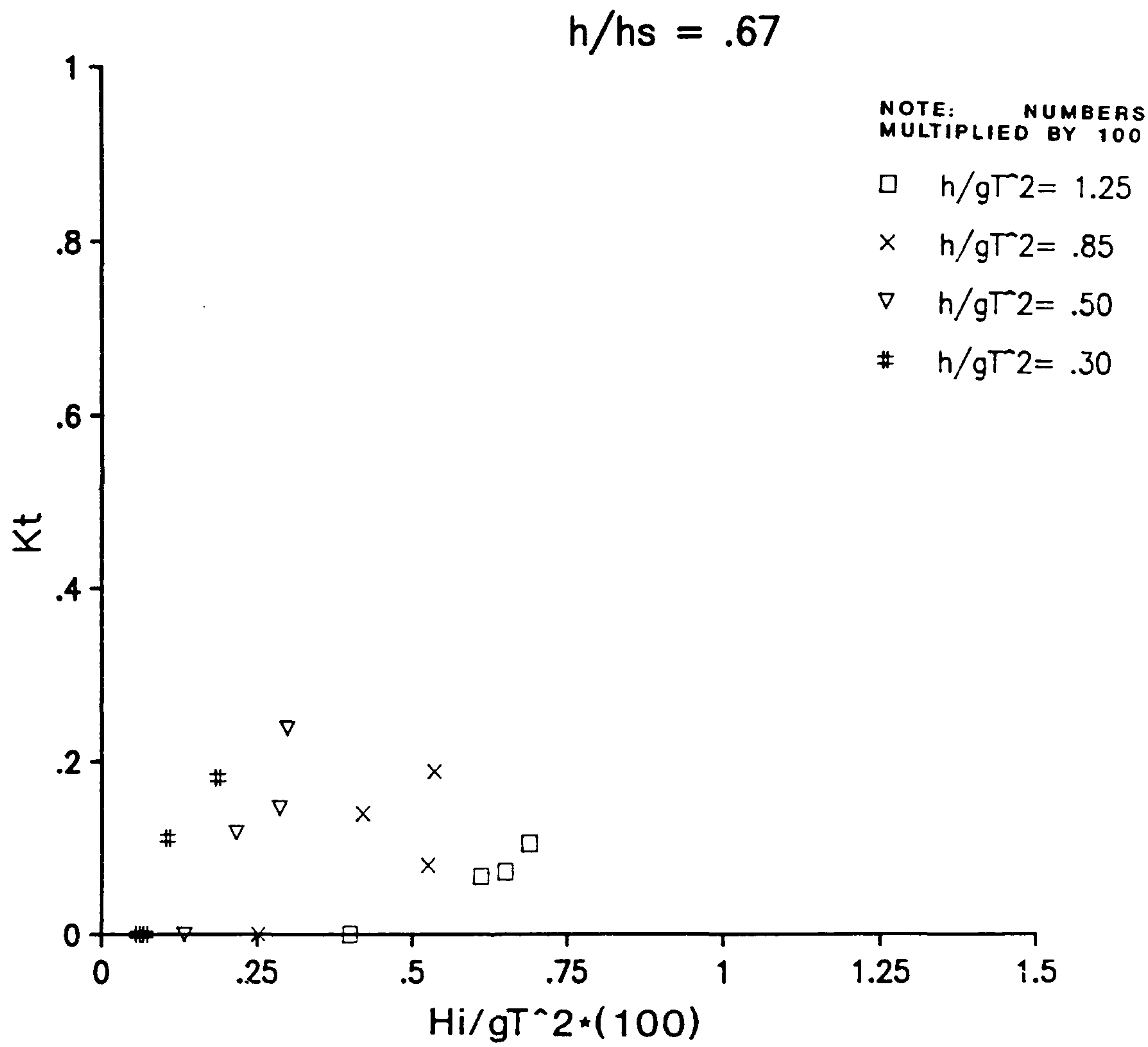


Figure 16. K_t versus wave steepness for solid breakwater

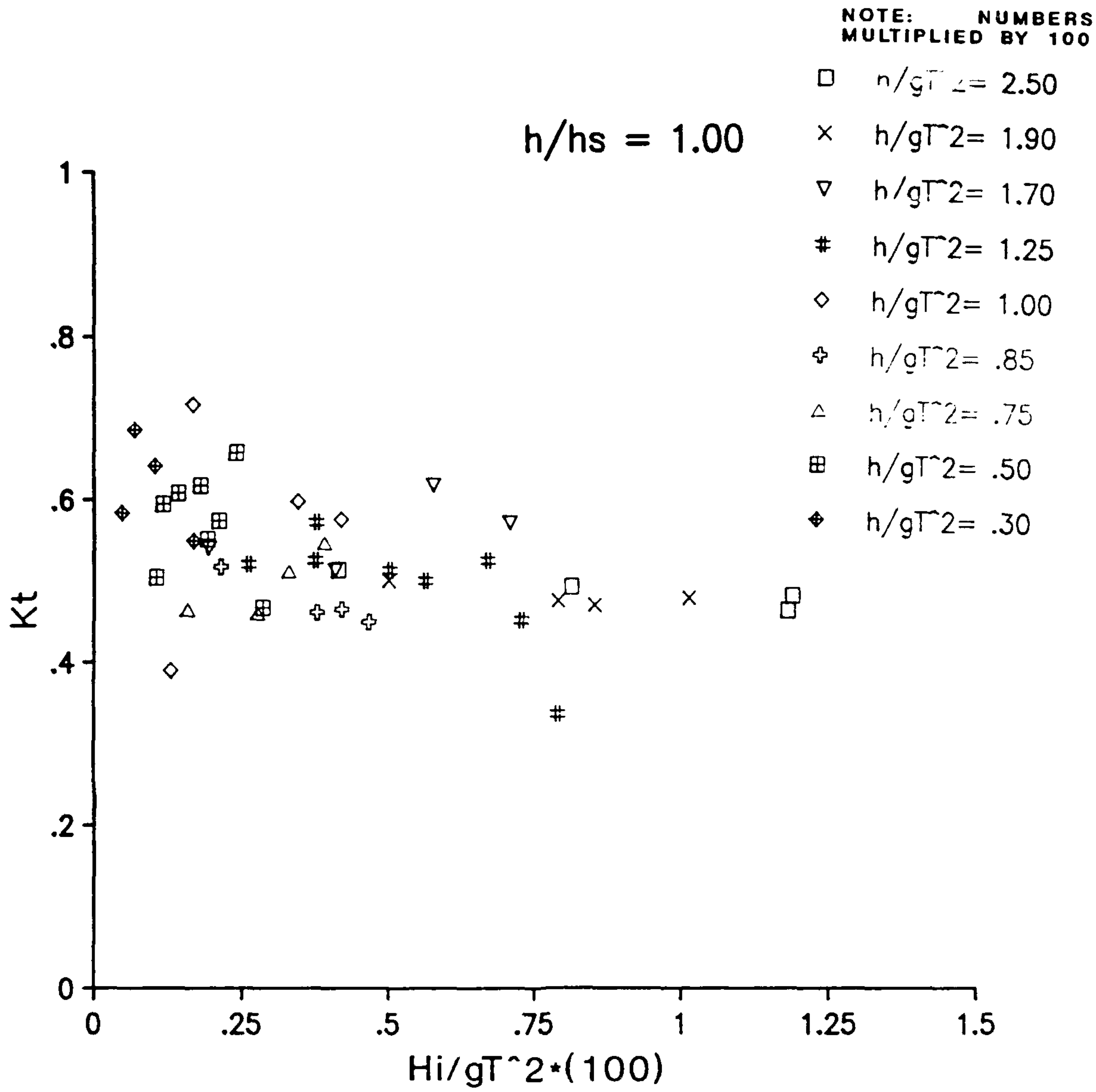


Figure 17. K_t versus wave steepness for solid breakwater

in this case is just the opposite to that shown for the previous case of very low breakwaters. In this case, considering that the structure is impermeable, no wave energy will overtop the breakwater for very low steepness waves, as can be seen by the transmission coefficients which are equal to zero. However, as the steepness of the waves increases, the waves will begin to overtop the breakwater through runup, and non-zero values for K_t are witnessed. The interesting trend in this case occurs as the wave steepness continually increases. It appears that the values for K_t approach a value of 0.4 to 0.5. This was also the case for h/h_s values greater than 1, or breakwaters with a negative freeboard.

The final category of breakwaters is shown in Figure 17. These conditions are the zero freeboard case where values of h/h_s equal 1. This breakwater condition was the most heavily studied of the three. At first, it appears that there is very little trend in this case as compared to the previous two cases. However, upon closer inspection, this case fits well within the trend established by the first two cases. As the wave steepness parameter increases, the transmission coefficient again decreases and approaches a value of about 0.5. In addition, as the water depth is varied, the values for K_t will also change. As can be seen, for low values of h/gT^2 and for low steepness waves, the transmission is greater than 0.5. But, as the steepness of the waves increases, no matter what the water depth, the value of K_t once again approaches a value of 0.5. In conclusion, Figures 13-17 show that values for K_t depend very heavily on wave steepness and breakwater geometry, and to a more limited degree on the relative water depth parameter, h/gT^2.

In addition to wave steepness, relative freeboard was investigated as a controlling parameter for wave transmission. The relative freeboard, F/H_i, was the next parameter studied, and values of K_t as a function of relative freeboard are shown in Figure 18. Relative freeboard has been the most widely investigated of any parameter. Goda (1969), Seelig (1980), and Ahrens (1987a, 1987b) are just a few of the authors who attribute transmission past a breakwater to this parameter. As illustrated by the data in Figure 18, when the relative freeboard of a structure is negative, or the structure is below the waterline, the incident waves are able to transmit overtop of the breakwater relatively easily, as seen by a K_t value very close to one. On the other hand, when the relative freeboard of the structure is positive, or the structure is above the waterline, the transmission coefficient is relatively low, and in some cases is actually zero. This is once again due to the fact that the structure, being solid, only allows the transmission of relatively large incident waves overtop.

At first it appears that relative freeboard does a good job in demonstrating trends in the data, however there is one major problem with this parameter. This problem occurs at

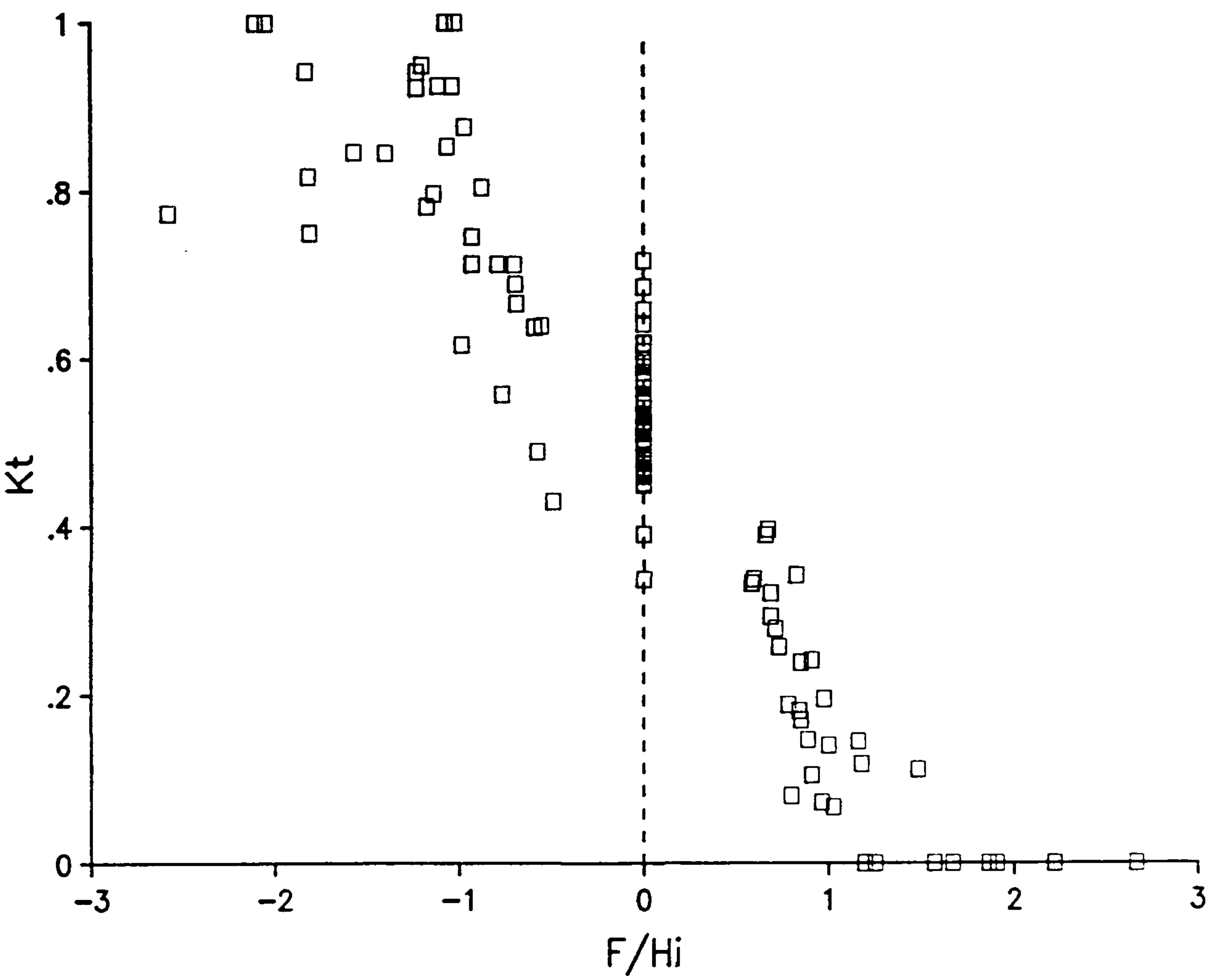


Figure 18. K_t versus relative freeboard for solid breakwater

relative freeboard values equal to zero, which was the case for a great many of the impermeable models studied. For the condition where $F/H_i = 0$, the values for K_t, varied from approximately 0.4 to 0.8, and are all located on the $F/H_i = 0$ axis. In other words, this parameter does not do a very good job in discriminating the various values of K_t at different water depths or wave steepness values.

From a practical engineering standpoint, this parameter is therefore not very effective. For example, if an engineer desired to build a reef breakwater with a freeboard at the waterline, or at a relative freeboard value of zero, he/she would have a very difficult time in predicting the value of K_t for that breakwater. However, as stated earlier, this parameter has been investigated extensively, and thus deserves some more discussion.

Goda (1969) proposed the following empirical equation for predicting wave transmission past a breakwater as a function of relative freeboard:

$$K_t = 0.5\{1-\sin[\frac{\pi}{2\alpha}(\frac{h_s-h}{H_i}+\beta)]\} \qquad \text{Eqn (3)}$$

In equation (3), the values of $\alpha = 2.6$ and $\beta = 0.15$ were used, as suggested by Goda (1969), and the resulting curve is shown in Figure 19. Goda's equation represents the trend in data well for the solid breakwater, although it overpredicts in some areas, and underpredicts in others. However, there still

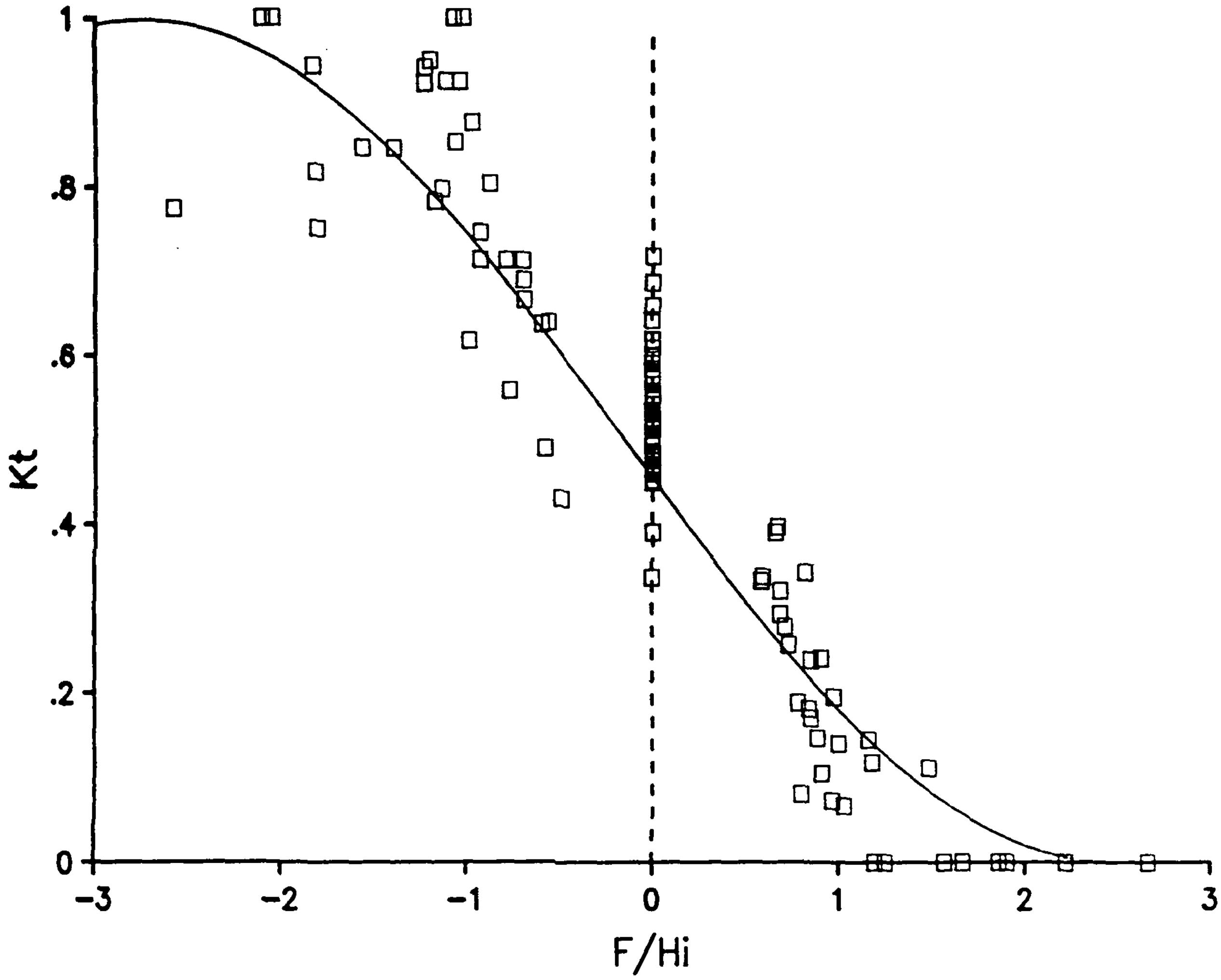


Figure 19. Comparison of Goda (1969) prediction method to data

exists the problem with values for relative freeboard equal to zero. What is being ignored in the foregoing is wave runup. Runup is an important parameter because it depends on wave height and period and, as a result, it varies with wave steepness.

Next, several different parameters were experimented with in an attempt to better represent the trends in wave transmission. One of these parameters was F/Ru, or freeboard divided by the runup on the structure, as suggested by Seelig (1980). Runup, however, was not a parameter that was measured in this study and all values used in this study are based on predictive equations given in Seelig (1980). However, there are many equations proposed for runup, and thus it important to decide which equation to use in a parameter defining wave transmission.

The first runup equation investigated was the Hunt runup formula, as given by Bruun (1989), but this was subsequently discarded because it is valid only for runup on small slopes, and does not work well for the steep slopes used in this study. Next, an equation for runup proposed by Franzius and given by Seelig (1980) for runup on solid impermeable slopes was investigated:

$$Ru=H_iC_1(0.123\frac{L}{H_i})^{(C_2\sqrt{\frac{H_i}{h}}+C_3)} \qquad \text{Eqn (4)}$$

where L is the local wavelength determined from Airy linear wave theory using the equation:

$$L=\frac{gT^2}{2\pi}\tanh\left(\frac{2\pi h}{L}\right) \qquad \text{Eqn (5)}$$

and C_1, C_2, and C_3 are empirical coefficients, which vary with breakwater geometry, as given in Seelig (1980). In this study, the values of these coefficients used for a slope of 1:1.5 were: C_1= 1.991, C_2= 0.498, and C_3= -0.185. Figure 20 shows K_t as a function of F/Ru, where runup was determined from equation (4). At first glance, this method of data representation looks very similar to the relative freeboard method, and in fact the same problem still exists when the freeboard of the breakwater studied is zero.

Seelig (1980) proposed the following empirical relationship relating F/Ru to K_t for smooth impermeable breakwaters with a 1:15 sloping beach in front of the breakwater cross-section:

$$K_{t_o}=C\left(1-\frac{F}{Ru}\right)-(1-2C)\frac{F}{Ru} \qquad \text{Eqn (6)}$$

where:

$$C=0.51-\frac{0.11B}{h_s};\ 0\leq\frac{B}{h_s}\leq 3.2 \qquad \text{Eqn (7)}$$

and B is defined as the crest width of the breakwater, or 6 inches in all cases. The first term of equation (6) is used for cases when F/Ru > 0 and both terms of equation (6) are used when F/Ru < 0. Transmission coefficients, K_t, are shown in Figure 21 as a function of F/Ru, where Ru is given by

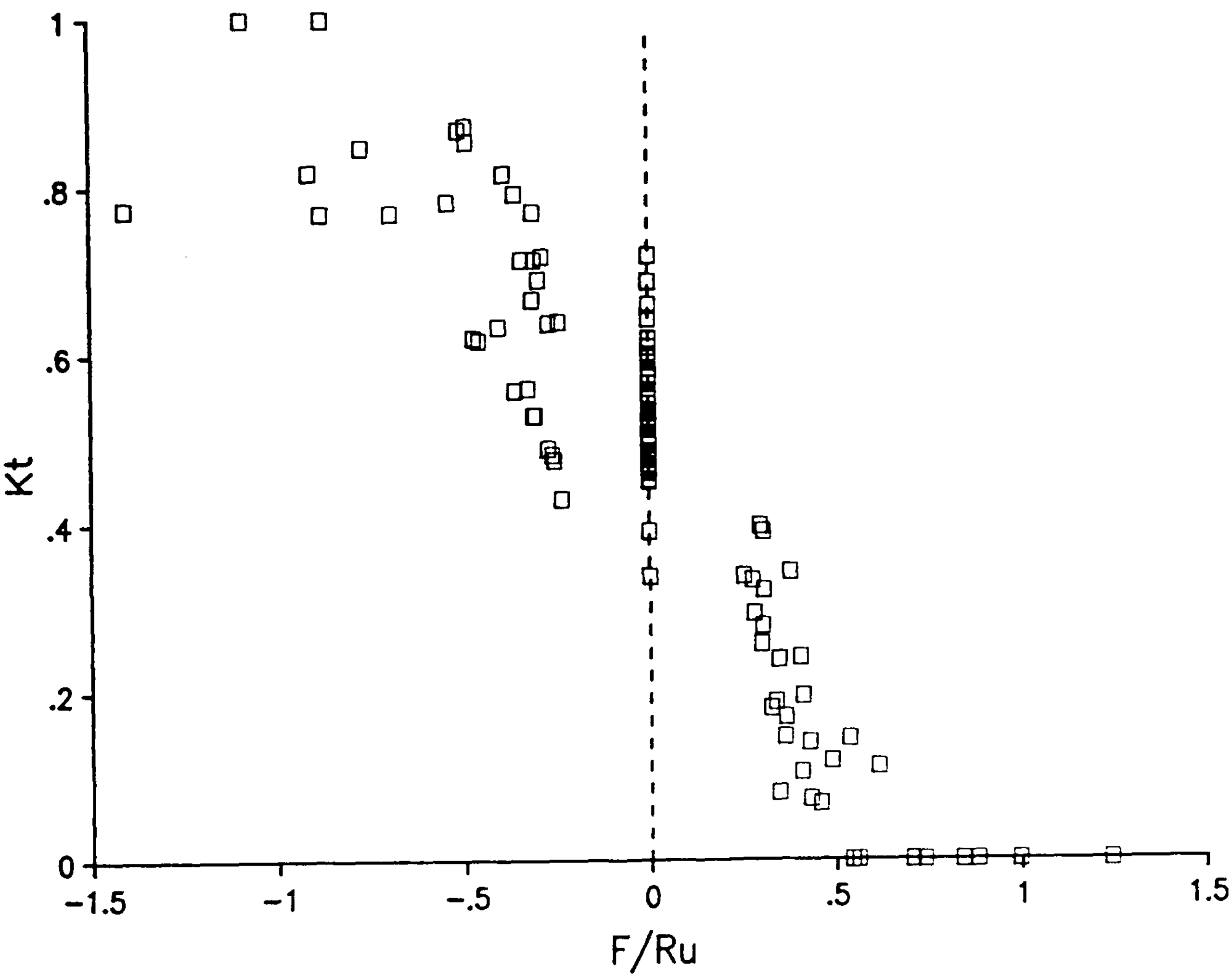


Figure 20. K_t versus F/Ru for solid breakwater

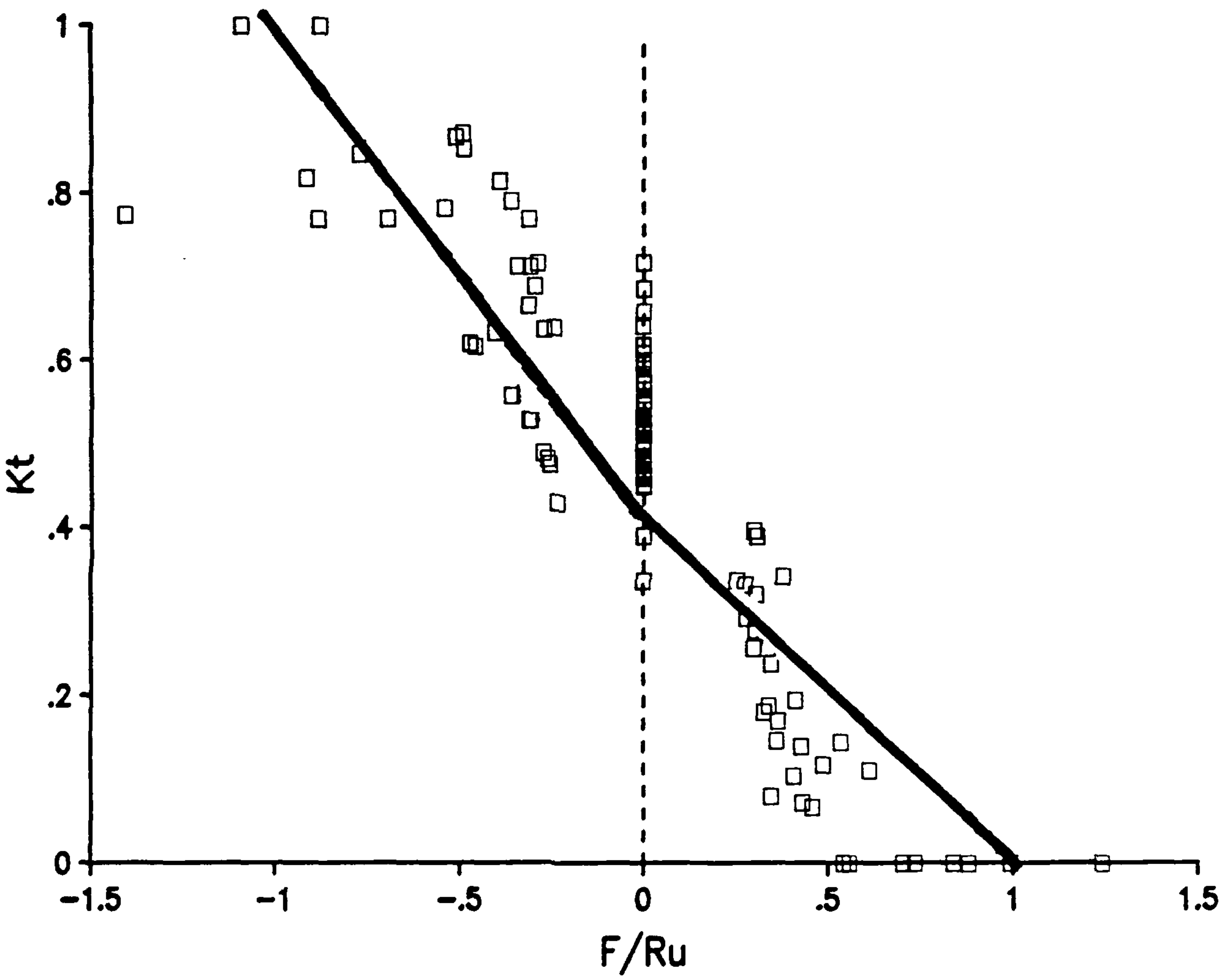


Figure 21. Comparison of Seelig (1980) prediction method to data

equation (4), with the empirical prediction equation (6) as proposed by Seelig (1980) shown over the data. As can be seen in Figure 21, Seelig's empirical equation, like Goda's, does a good job in predicting the trend in the transmission data, with the exception that it underpredicts when F/Ru is equal to zero.

Following the investigation of the F/Ru and relative freeboard parameters, several other parameters were investigated. It was hoped that a new parameter could be found that would demonstrate the trend in the data without the problems associated with the zero freeboard condition. The most successful parameter investigated was the $(F\text{-}Ru)/H_i$ parameter. This parameter was selected because it includes the effect of wave steepness through the presence of runup in the equation, and even at zero freeboard conditions, all test conditions have different values of runup. In addition, as the F-Ru parameter approaches zero, it has a physical lower bound where K_t must equal zero since the wave runup cannot overtop the breakwater.

In Figure 22, K_t is shown as a function of this parameter where equation (4) is used to determine the runup in the parameter. As can be seen, roughly the same trend is present in the data as was the case for the first two parameters investigated. For cases where the potential runup is much greater than the freeboard, the values of K_t approach 1. This can be attributed to the fact that the incident wave is able

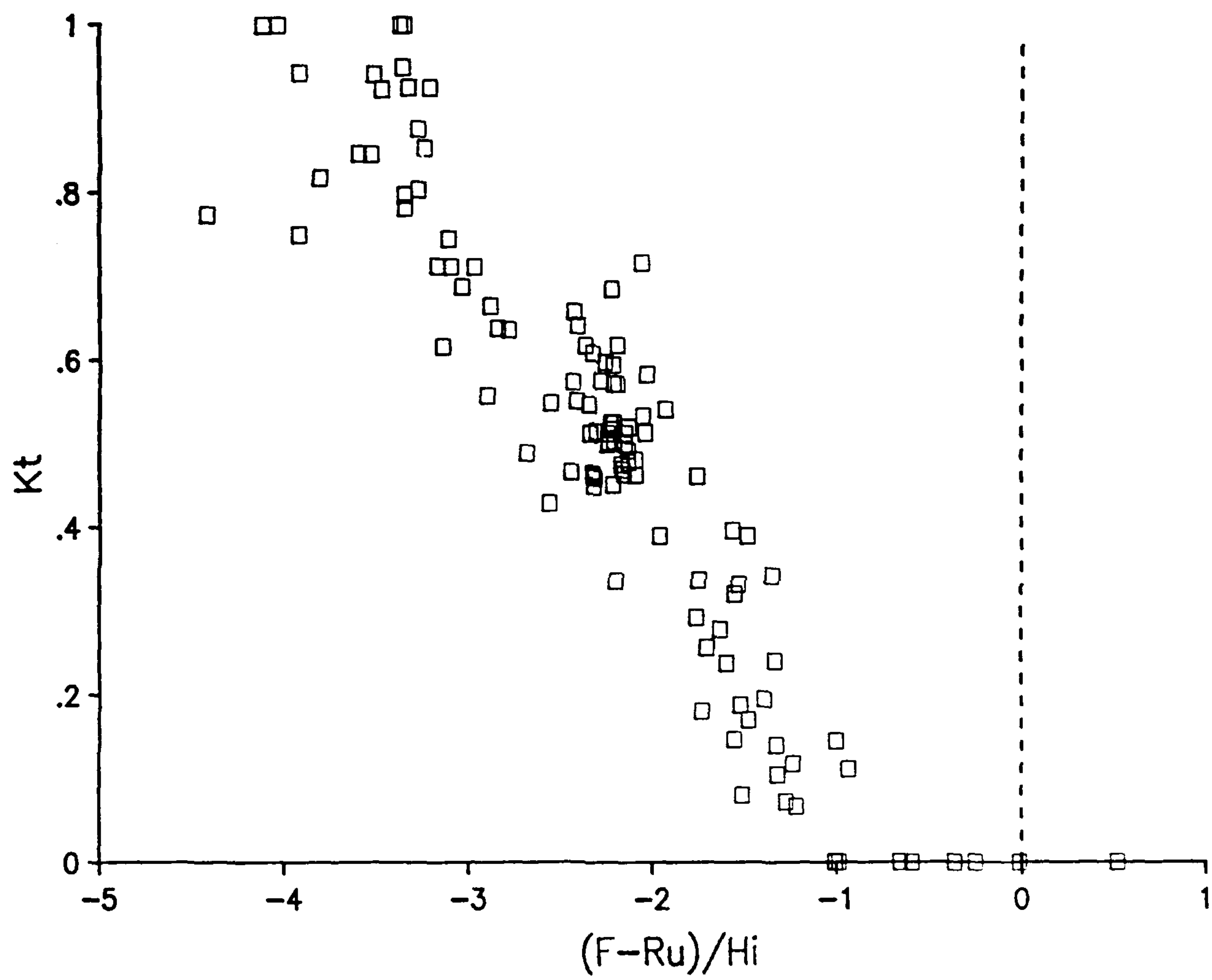


Figure 22. K_t versus $(F\text{-}Ru/H_i)$ for solid breakwater using Ru from Seelig (1980)

to overtop the breakwater easily. On the other end, due to the fact that this is a solid breakwater, when the freeboard is equal to or greater than the runup, the transmission coefficient should be zero. However, in Figure 22, the transmission coefficient approaches zero while the runup is still greater than the freeboard, which is not physically consistent. Thus, because the runup part of the parameter is generated from a predictive equation, it is necessary to investigate another runup formula in hopes of representing the data better at values when F-Ru approaches zero.

The next runup equation investigated was proposed by Ahrens and McCartney (1975). This equation relates runup, Ru, to the surf similarity parameter, ξ:

$$\frac{Ru}{H_i} = \frac{a\xi}{1+b\xi}; \quad \xi = \frac{\tan\theta}{\sqrt{H_i/L_o}} \qquad \text{Eqn (8)}$$

where L_o is the deep water wavelength or $gT^2/2\pi$ and a and b are empirical coefficients which have the values of a= 0.775 and b= 0.361, as proposed by Gunbak (1979). Values of K_t are again shown as a function of $(F-Ru)/H_i$ in Figure 23, but this time with the runup equation (8). As can be seen in Figure 23, equation (8) does a better job of representing the data than equation (4), especially in the region where F-Ru approaches zero. Unlike Figure 22, Figure 23 shows that values of K_t equal to zero only occur for values of F-Ru which are positive or very close to zero as expected. Considering

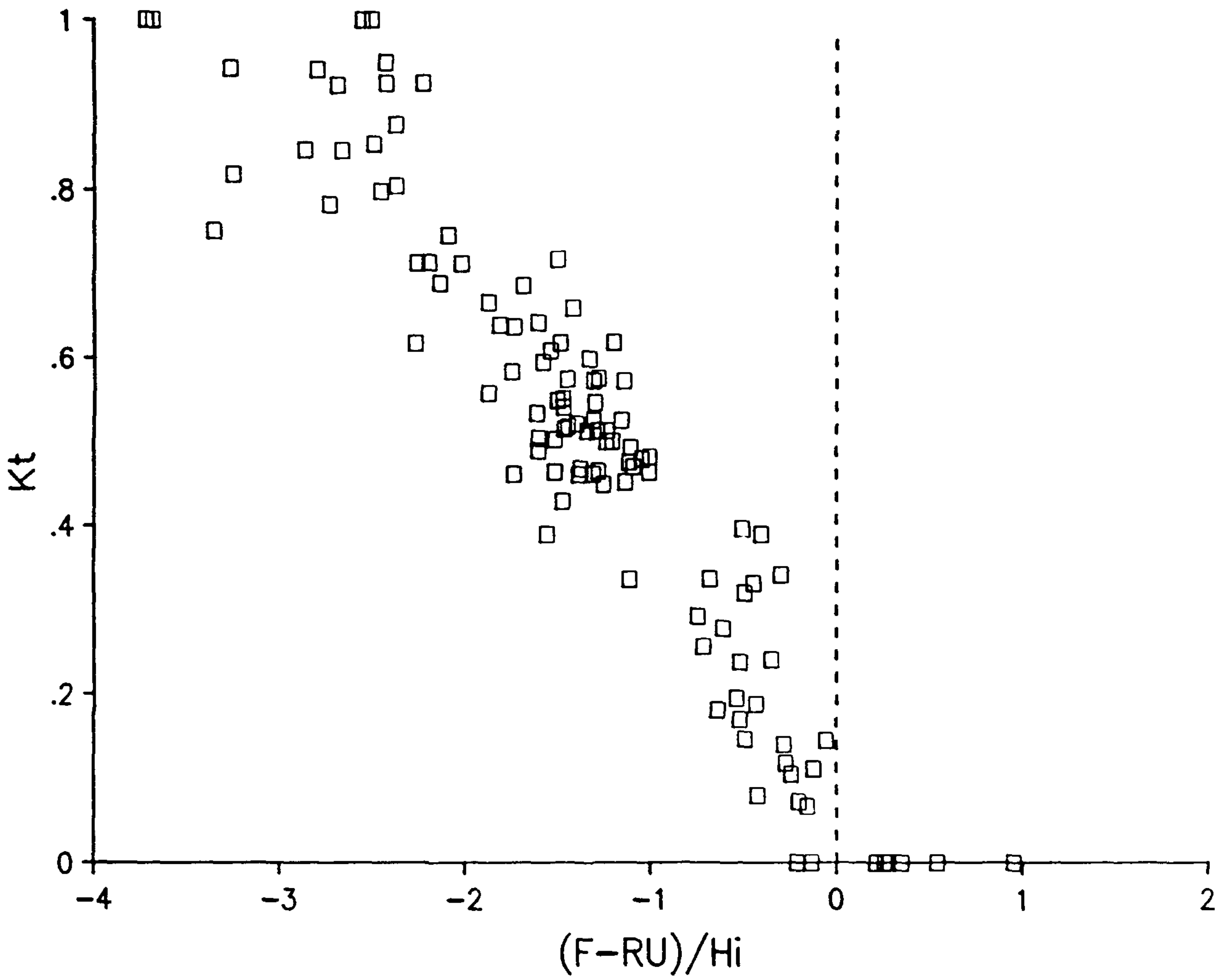


Figure 23. K_t versus (F-Ru)/H_i for solid breakwater using Ru from Gunbak (1979)

this, and the manner in which this equation predicted runup on rubble structures as will be shown later, it was decided that equation (8) with the coefficient values proposed by Gunbak (1979) should be used to estimate the runup on the breakwater cross sections for the remainder of the study.

In conclusion, this new parameter, $(F-Ru)/H_i$, does a very good job in representing the appropriate trends in the data. In addition, and perhaps of even more importance from an engineer's point of view, this new parameter does not suffer from the same problems as did the relative freeboard, F/H_i, and F/Ru parameters. In other words, this new parameter is capable of discriminating wave transmission coefficients for all types of breakwater freeboards, including those equal to zero.

Figure 24 shows an empirical relationship which is proposed to represent the trends shown with this parameter. In essence, the proposed equation is simply a straight line. It attempts to relate the physical aspects of the breakwater studied to the data gathered. The line starts where K_t equals 1 at very small values of $(F-Ru)/H_i$, or those values where the water depth plus runup is much greater than the height of the structure. The line then slopes down and eventually reaches zero at the point when the freeboard is equal to the runup. The slope and the point at which the line begins to slope were both determined exclusively from the data gathered. Although a straight line at first seems to be a very simple method of

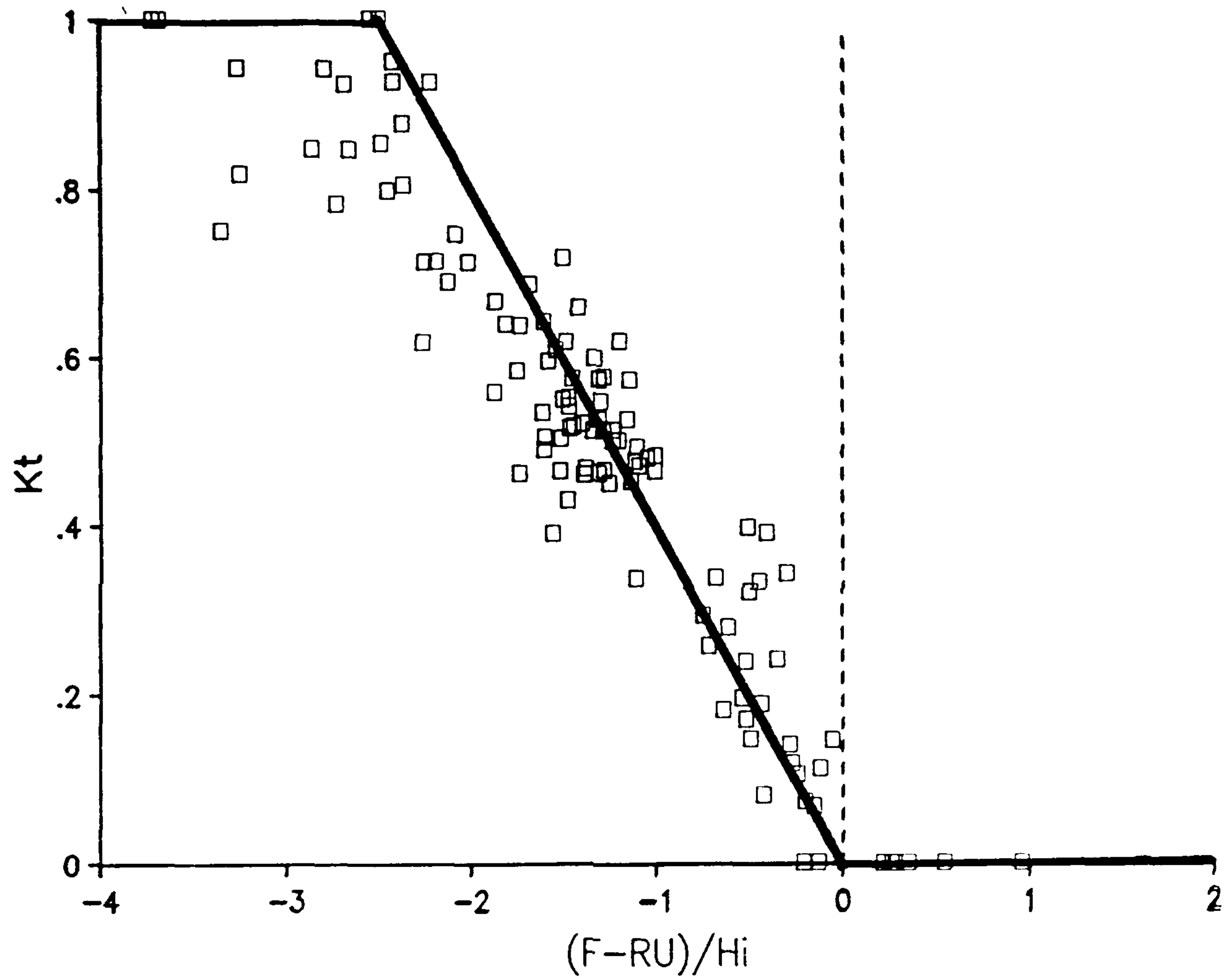


Figure 24. Comparison of proposed empirical equation to data

depicting the data gathered in comparison to the methods proposed by others, it must be considered that this is a new parameter, and indeed, the straight line method does in fact do just as well with this parameter as did Seelig's equation with F/Ru and Goda's with F/H_i.

RUBBLE BREAKWATER--REGULAR WAVES

The wave basin setup for this section of the study was exactly the same as for the solid breakwater tests just discussed, with the exception that permeable models, as shown in Figures 25 and 26 were used. Once again, as shown in Figure 7, three wave gages were used, two fixed gages behind the breakwater and one moving gage in front. The gages were placed the same distance in front of and behind the structure as were done in the solid portion of the study. The same method of data collection and analysis was performed once again. A strip chart recorder was used to measure the incident wave envelope and the transmitted wave heights. Once again, the wave heights of the two gages behind the breakwater were averaged to yield the transmitted height and the wave envelope in front of the breakwater was analyzed to yield the incident and reflected wave heights.

Initially, the same parameters used for the solid breakwater, regular wave section of the report were investigated in hopes of comparing the two groups of tests in

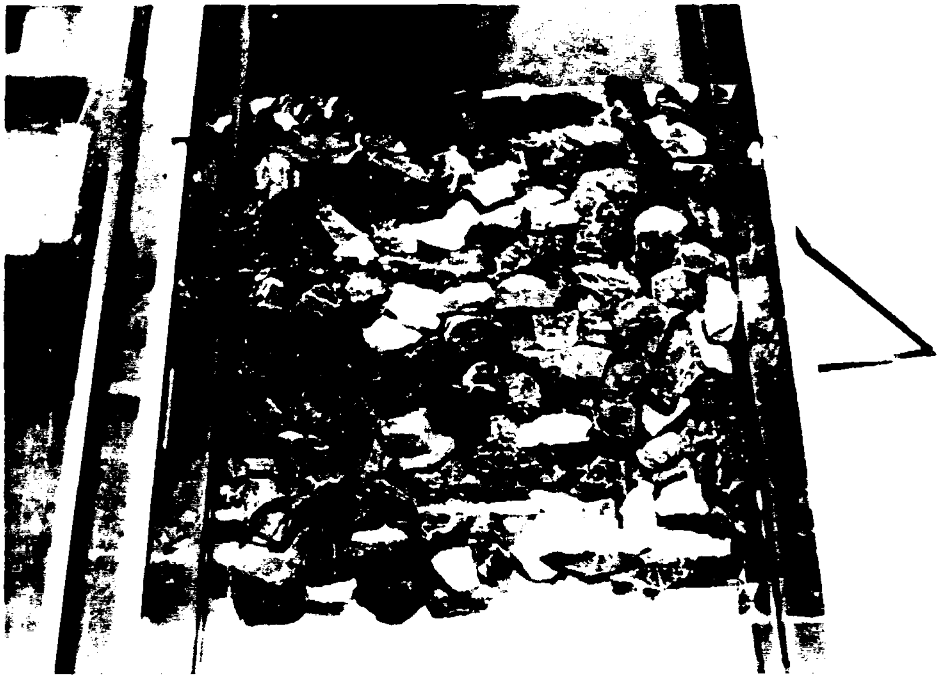


Figure 25. Top view of rubble cross-section used in lab.

Figure 26. Side view of rubble cross-section used in lab.

order to distinguish between transmission overtop and transmission through the breakwater. The first parameter investigated was therefore the wave steepness parameter, H_i/gT^2, as proposed by Seelig (1980). Once again this parameter is shown in Figures 27-31 as a function of K_t for various values of relative depth, h/gT^2.

Figures 27 and 28, show the trends in transmission for structures with a negative freeboard, where h/h_s is greater than one. The trend in these figures is very similar to the trend in Figures 13-14, for solid breakwaters. Because the breakwaters are below the waterline, the incident wave does not have difficulty in passing over the structure when the wave is of low steepness, and thus the transmission values are all very close to 1. However, as with solid breakwaters, when the wave steepness increases and the wave becomes more critical, the breakwater succeeds in "tripping" the incident wave causing it to break. This breaking of the wave dissipates a great deal of the incident wave energy and the values of K_t approach a value of 0.5 to 0.6, which is slightly higher than was the case for solid breakwaters.

The second type of breakwaters considered are those whose crest heights were greater than the water depth, with h/h_s values less than one, as shown in Figures 29 and 30. In this case, the trend of the data is much different than for the solid breakwaters studied earlier. Initially, for very low steepness waves there is transmission, whereas for the solid

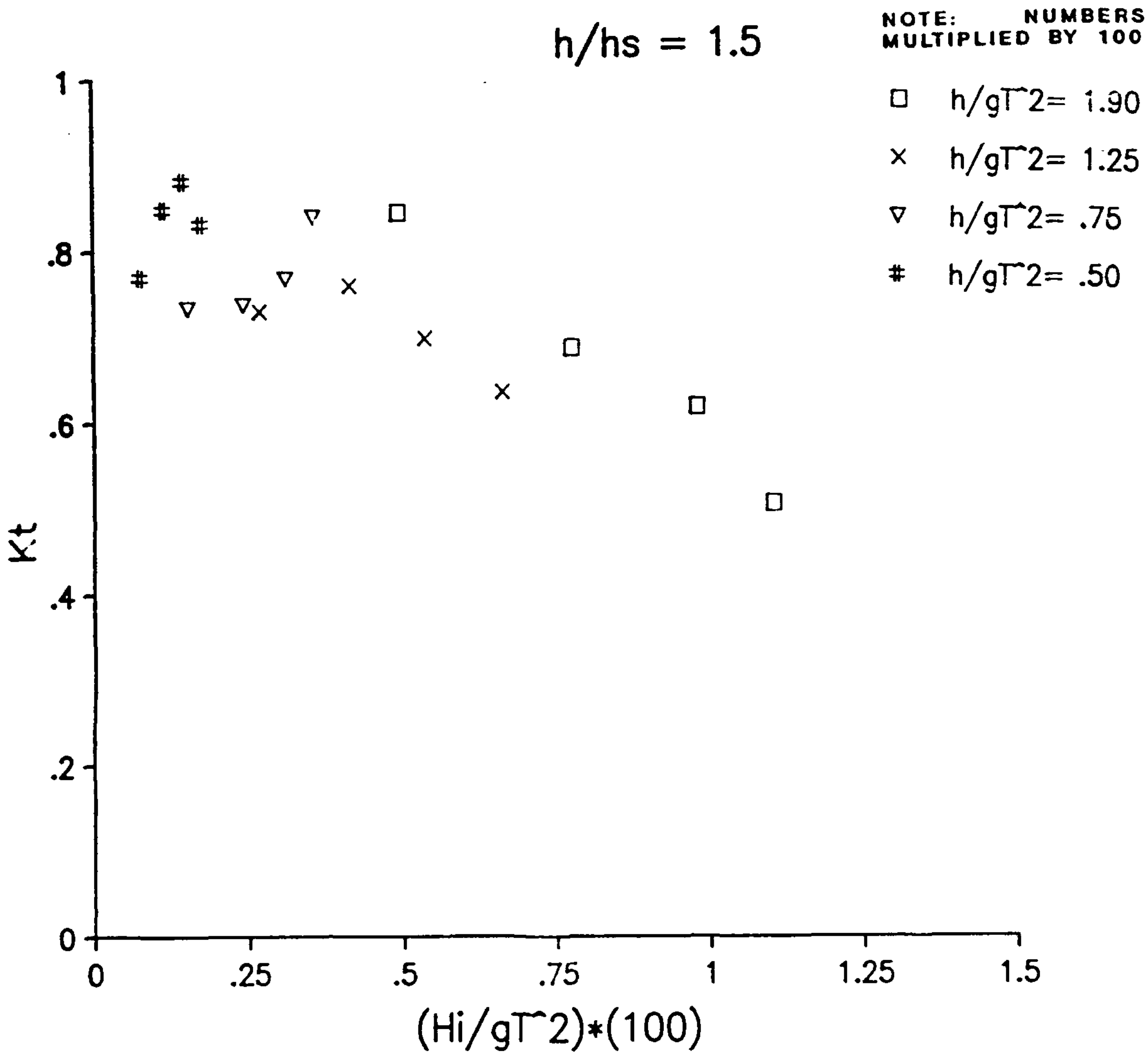


Figure 27. K_t versus wave steepness for rubble breakwater

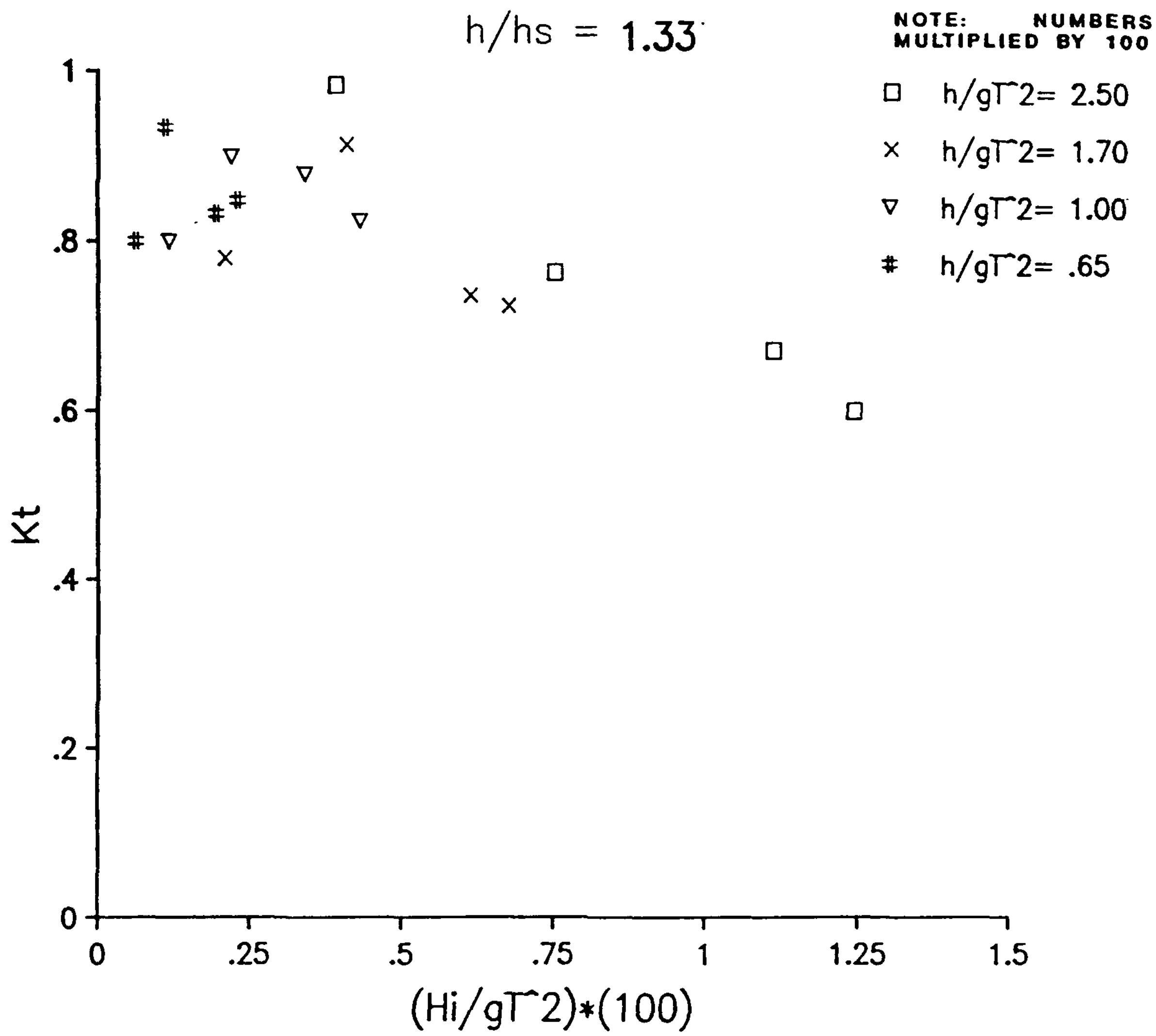


Figure 28. K_t versus wave steepness for rubble breakwater

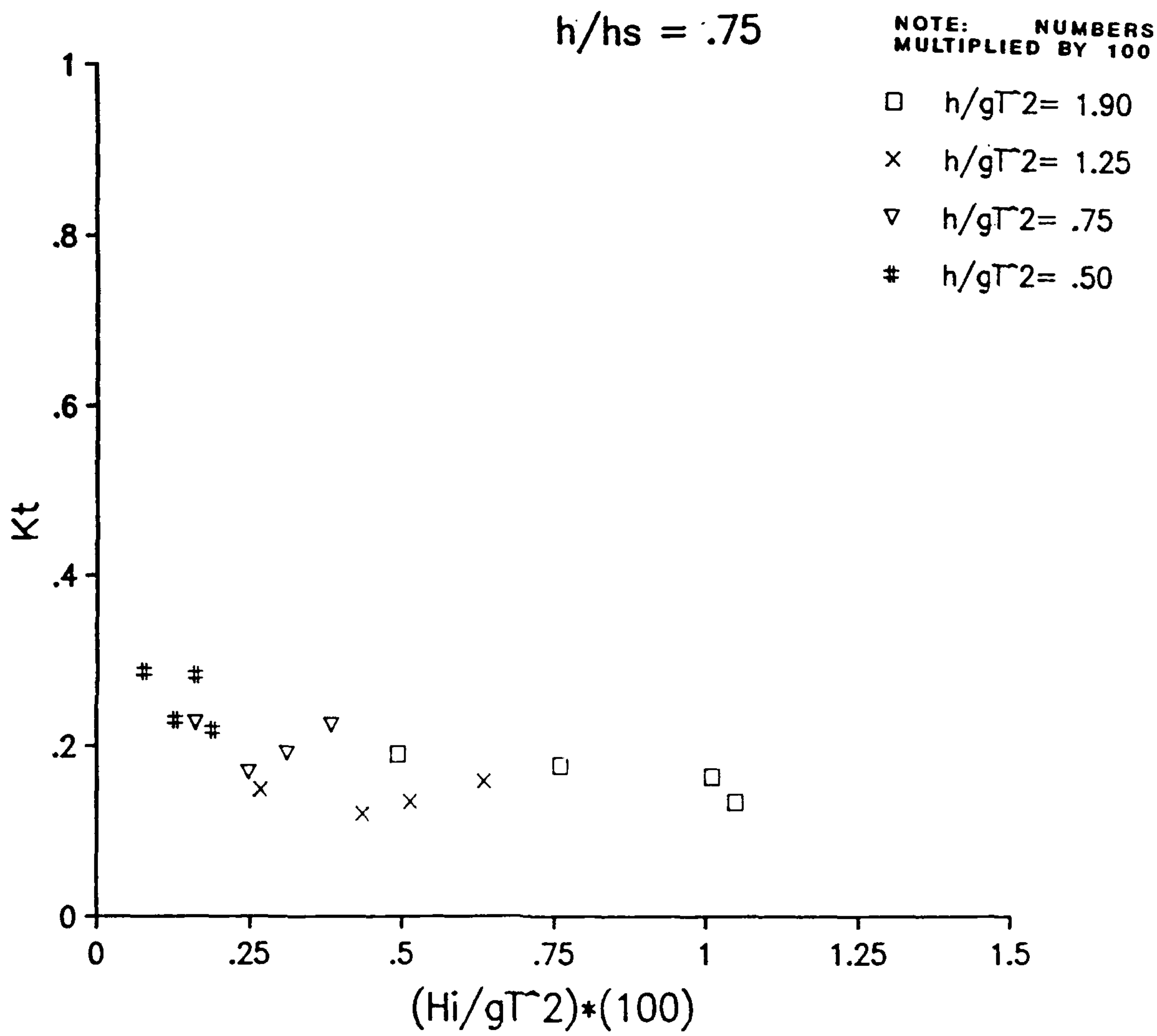


Figure 29. K_t versus wave steepness for rubble breakwater

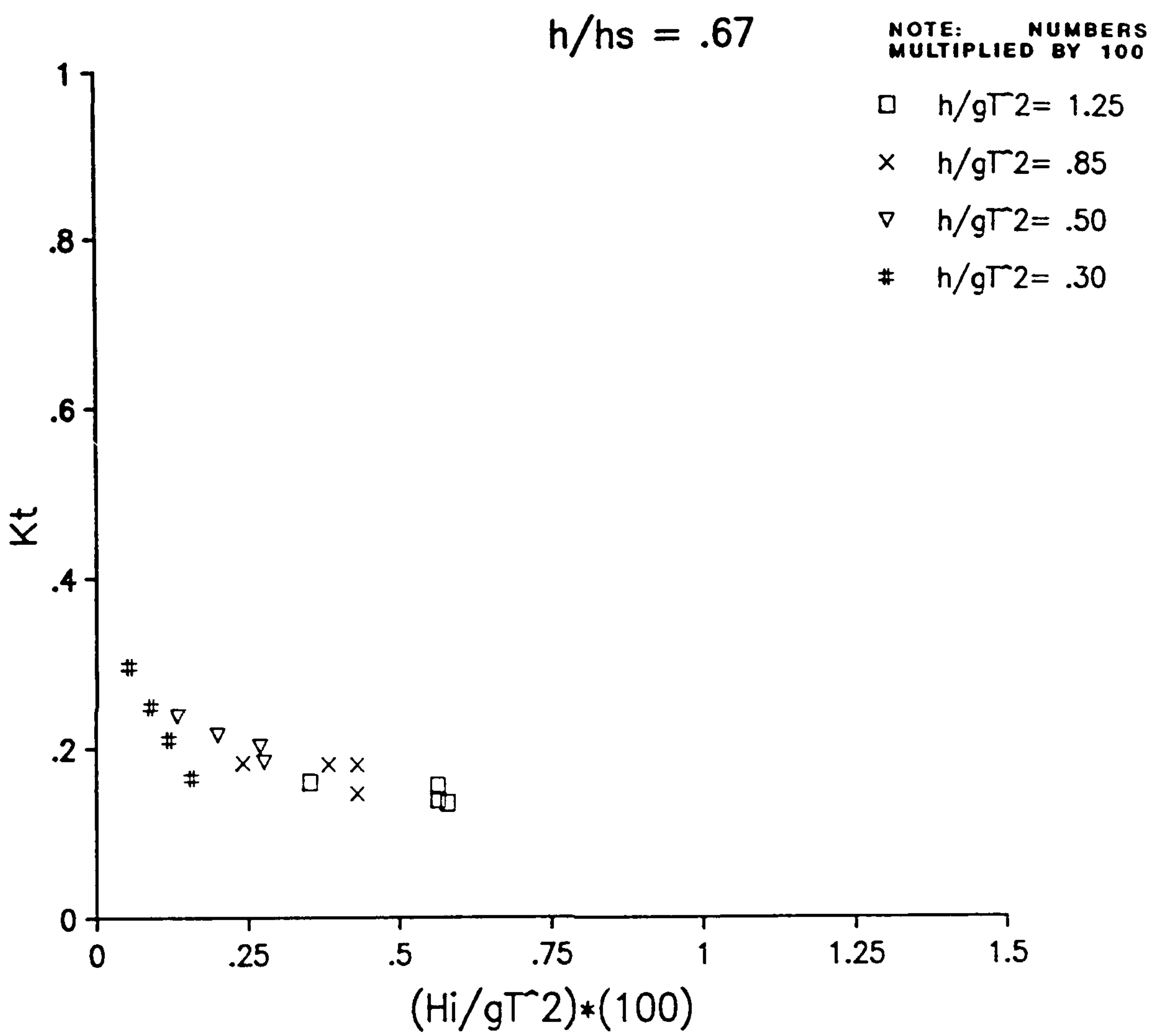


Figure 30. K_t versus wave steepness for rubble breakwater

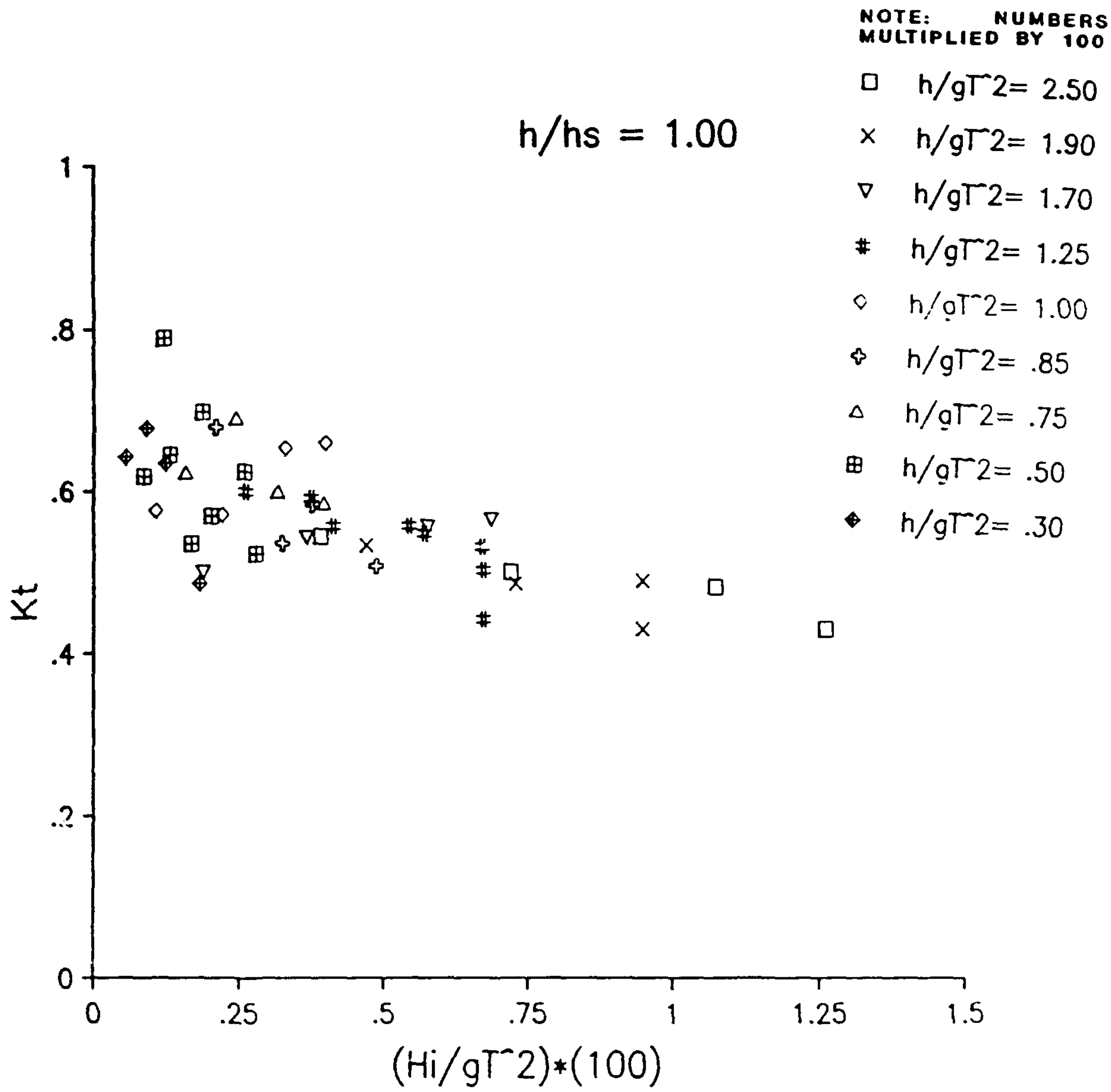


Figure 31. K_t versus wave steepness for rubble breakwater

breakwater there had been none. Thus, it can be concluded that all of the transmission for very low values of H_i/gT^2 is due to flow through the permeable breakwater. In addition, the data do not tend toward values of K_t equal to 0.5 as the wave steepness increases as was the case for the solid breakwaters. In the case of permeable structures, just the opposite is true. Instead of K_t increasing as wave steepness increases, it decreases. This is due to the fact that the breakwater is more effective in dissipating energy as waves propagate through the structure. In addition, the waves are also not as able to runup the surface of a rough permeable breakwater to the same degree that they were able to runup a smooth impermeable structure. Thus, much less of the incident wave is able to overtop the structure, instead it must pass through the pores of the structure. In the process of passing through the structure, the high steepness waves, in particular, are not able to pass through the breakwater as cleanly as are the low steepness waves. In the breakwater, energy dissipation is related to the water particle velocities squared, much like the head loss in turbulent pipe flow, and high steepness waves dissipate much more of their energy. This is the reason behind the downward trend in the data for high values of the wave steepness parameter.

The final category of breakwater, those with zero freeboards, is illustrated in Figure 31. Figure 31 is very similar to Figure 17, for the solid breakwaters. Once again, all of the data tend toward a K_t value of slightly less than 0.5. As was the case for the solid breakwater, the rubble breakwater allows greater transmission of the low steepness waves, while it attenuates the higher steepness waves. Overall, the data seem to converge on the same value for K_t as they did for solid breakwaters. Once again, this wave steepness parameter, as proposed by Seelig (1980), does a very good job in representing the expected trends in the data for rubble breakwaters just as it did for solid breakwaters.

The second parameter investigated was the relative freeboard parameter, F/H_i, as seen in Figure 32. The wave transmission as a function of the relative freeboard shows much of the same initial trend in the data as was seen for solid breakwaters, especially for the values of negative relative freeboard. The difference appears for the higher positive values of F/H_i. At these high values of relative freeboard, the values of K_t do not trend towards zero as they did with solid breakwaters, but instead they appear to trend slowly back toward 1. This can be attributed to the transmission through the structure.

Upon closer inspection of this phenomenon, what is interesting is that in this study, due to the fact that the highest value of freeboard was set at 2 inches, the controlling factor in the parameter at these high values of relative freeboard is the denominator, or the incident wave height. Thus, in order for the relative freeboard parameter

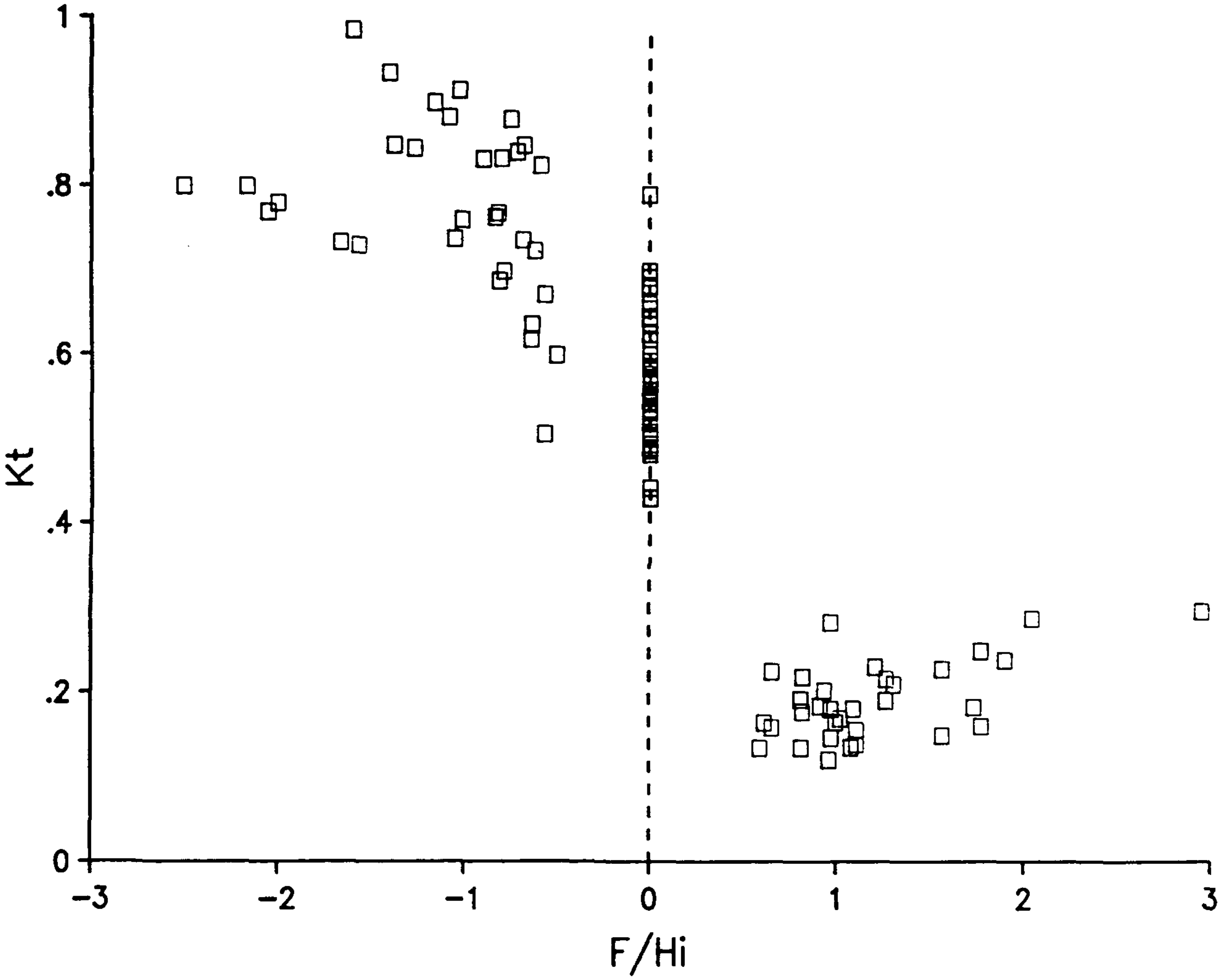


Figure 32. K_t versus relative freeboard for rubble breakwater

to be high, the incident wave height must be very small, or in other words, the wave must have very low steepness and must be very linear. As a result, this parameter demonstrates the same effect on wave transmission that the wave steepness parameter had done previously. This parameter illustrates that for very small steepness waves of a linear nature, the breakwater allows more and more transmission through the structure, even though the freeboard of the structure may be much higher than the incident wave.

The relative freeboard parameter, although it describes the data rather well, still suffers from the same shortcomings that were noticed with the solid breakwaters. In other words, it still does not separate the data at values of freeboard equal to zero and a design engineer would have a very difficult time using this parameter to design an actual breakwater. However, despite this shortcoming, this parameter does do a good job at displaying a minimum value of K_t, when F/H_i, where transmission overtop of the structure fails to dominate, and transmission through the structure begins to dominate.

Once again, as shown in Figure 33, the experimental data may be compared to prediction from Goda's empirical equation (3). Despite the fact that Goda (1969) proposed equation (3) for solid breakwaters, it still does a good job in predicting values for K_t below relative freeboard values equal to 1. Interestingly, the point after which Goda's equation fails to

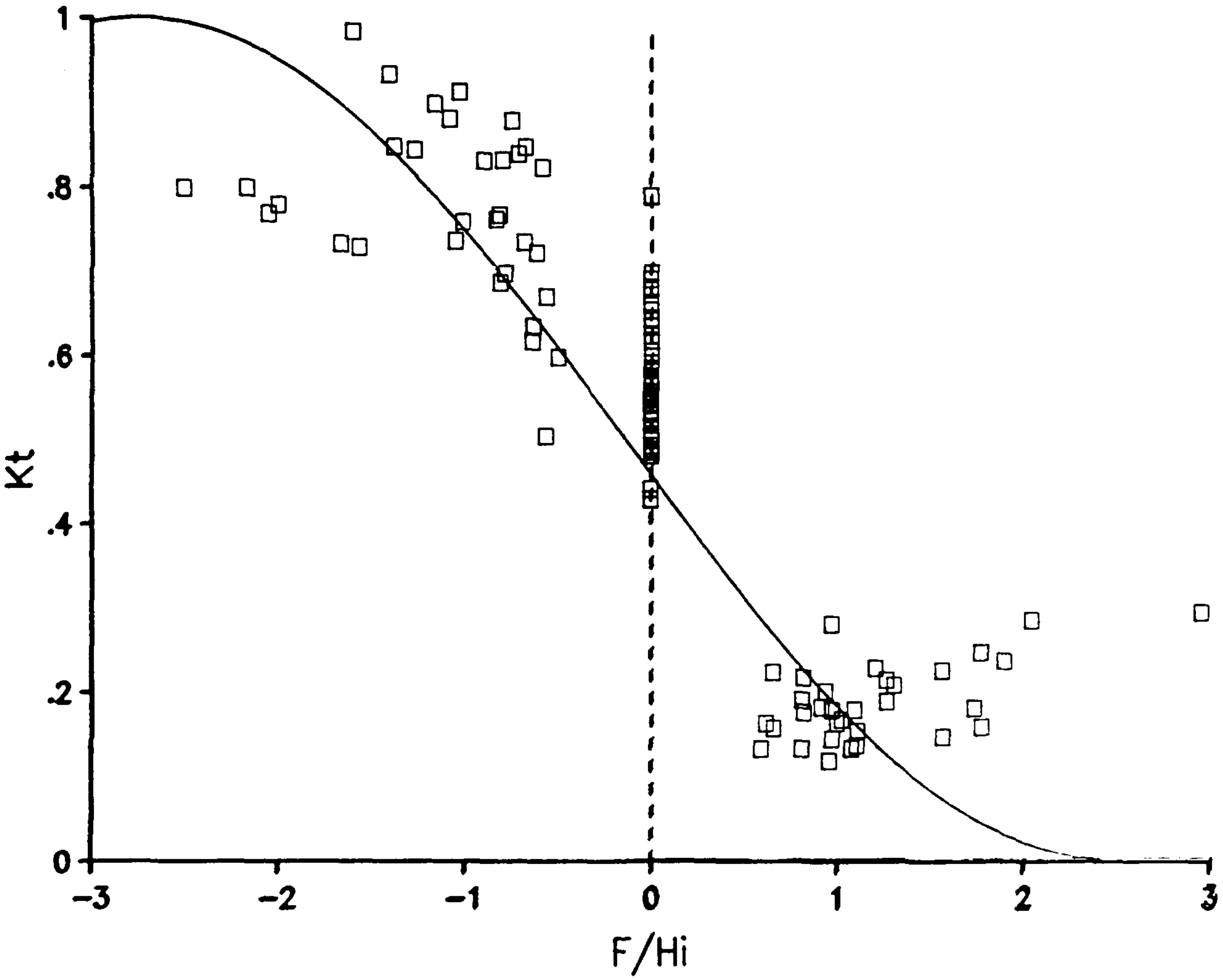


Figure 33. Comparison of Goda (1969) prediction method to data for rubble breakwater

predict values of K_t accurately is the minimum point in the data as was discussed above. Goda's equation further reinforces the fact that at this minimum point of $F/H_i = 1$, the values for K_t cease from being dominated by overtopping to being dominated by transmission through the structure.

The third parameter investigated was the F/Ru parameter as shown in Figure 34. Runup, in this parameter, was determined from equation (8) with the coefficients provided by Gunbak (1979). Once again, the same trend is seen for the data as was illustrated with the relative freeboard parameter. It is again easy to see where transmission through the structure dominates ersus transmission overtop. In fact, the same minimum exists, and the data appear to trend back towards a K_t value of one, as the F/Ru reaches larger and larger values. Once again this can be attributed to the same factors as were previously discussed concerning relative freeboard. Namely, as the runup, which is a function of wave steepness, becomes smaller and smaller the structure is less and less effective in dissipating the incident wave energy. Thus, the values for K_t will increase, again theoretically approaching a value of one for very low steepness waves, such as tides for example. However, the practicality of this parameter is still lacking due to its failure to discriminate the data well for freeboards equal to zero.

The fourth parameter investigated was the $(F-Ru)/H_i$ parameter. Values of K_t as a function of this parameter are

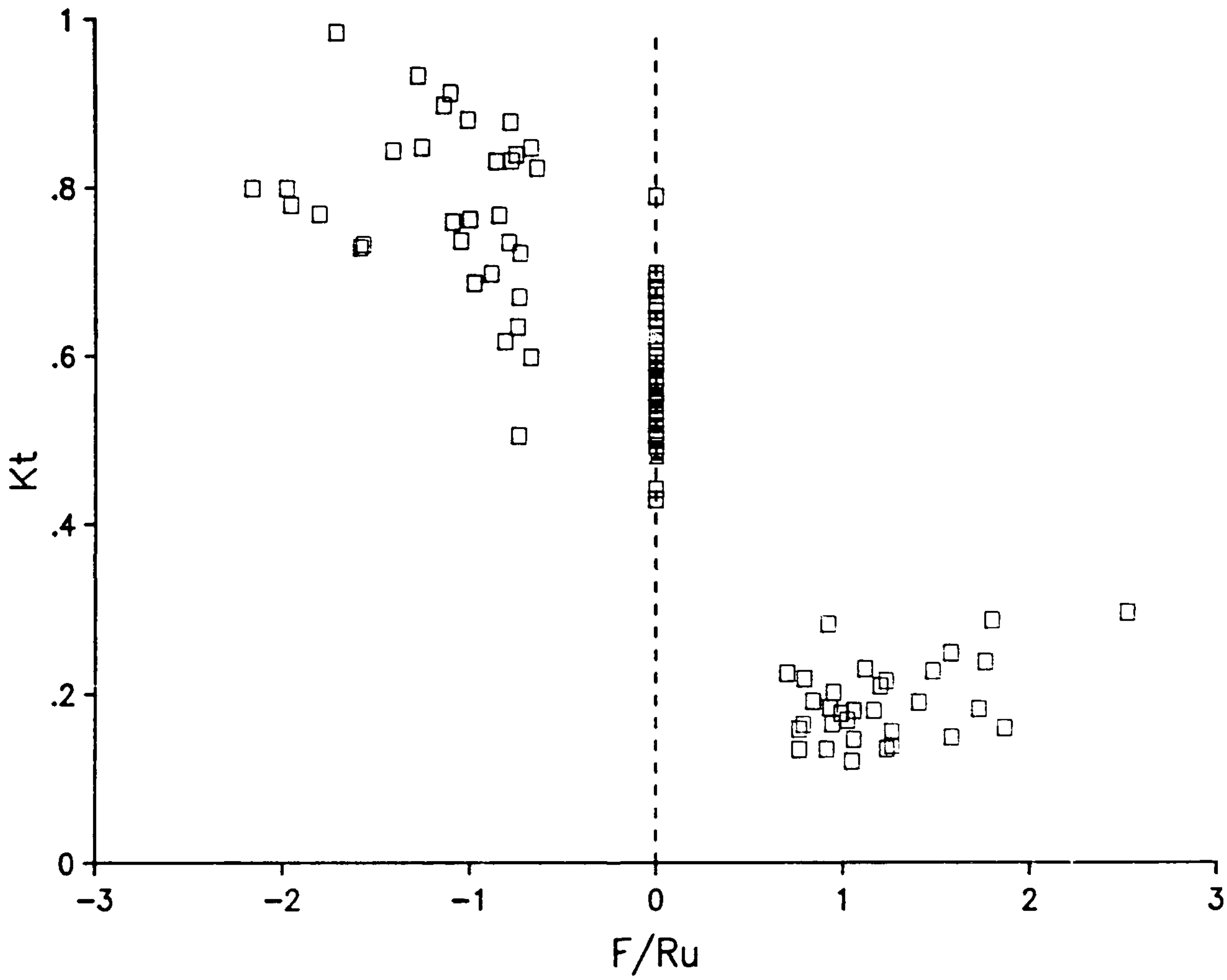


Figure 34. K_t versus F/Ru for rubble breakwater

shown in Figure 35. As with the earlier parameters of F/H_i and F/Ru, this parameter shows the same trend in the data, with the exception that it does not group the data at values of freeboard equal to zero. Thus, this parameter would be more acceptable for use in proposing the design of an actual reef breakwater, especially one with a freeboard close to the waterline. In Figure 35, K_t values are once again near 1 for small negative values of $(F-Ru)/H_i$ and decrease almost linearly as $(F-Ru)/H_i$ approaches zero. Then, for positive values of $(F-Ru)/H_i$, K_t again increases as low steepness waves propagate through the structure.

Figure 36, shows three empirical relationships which attempt to describe the data under different circumstances. The first curve, which was generated earlier for solid breakwaters and shown in Figure 24, starts at one and trends to zero when the freeboard equals the runup. This curve attempts to predict the values of K_t which are solely a product of the transmission overtop of the structure. It is interesting to note that this curve overlays the data very well for values of $(F-Ru)/H_i$ less than -1. On the other hand, the second curve which starts at zero and trends back towards one for infinitely large values of the parameter attempts to predict the values of K_t which are solely a product of the transmission through the structure. Finally, the third curve is a combination of the first two. The third curve is based on adding the energy of two wave systems together. It was

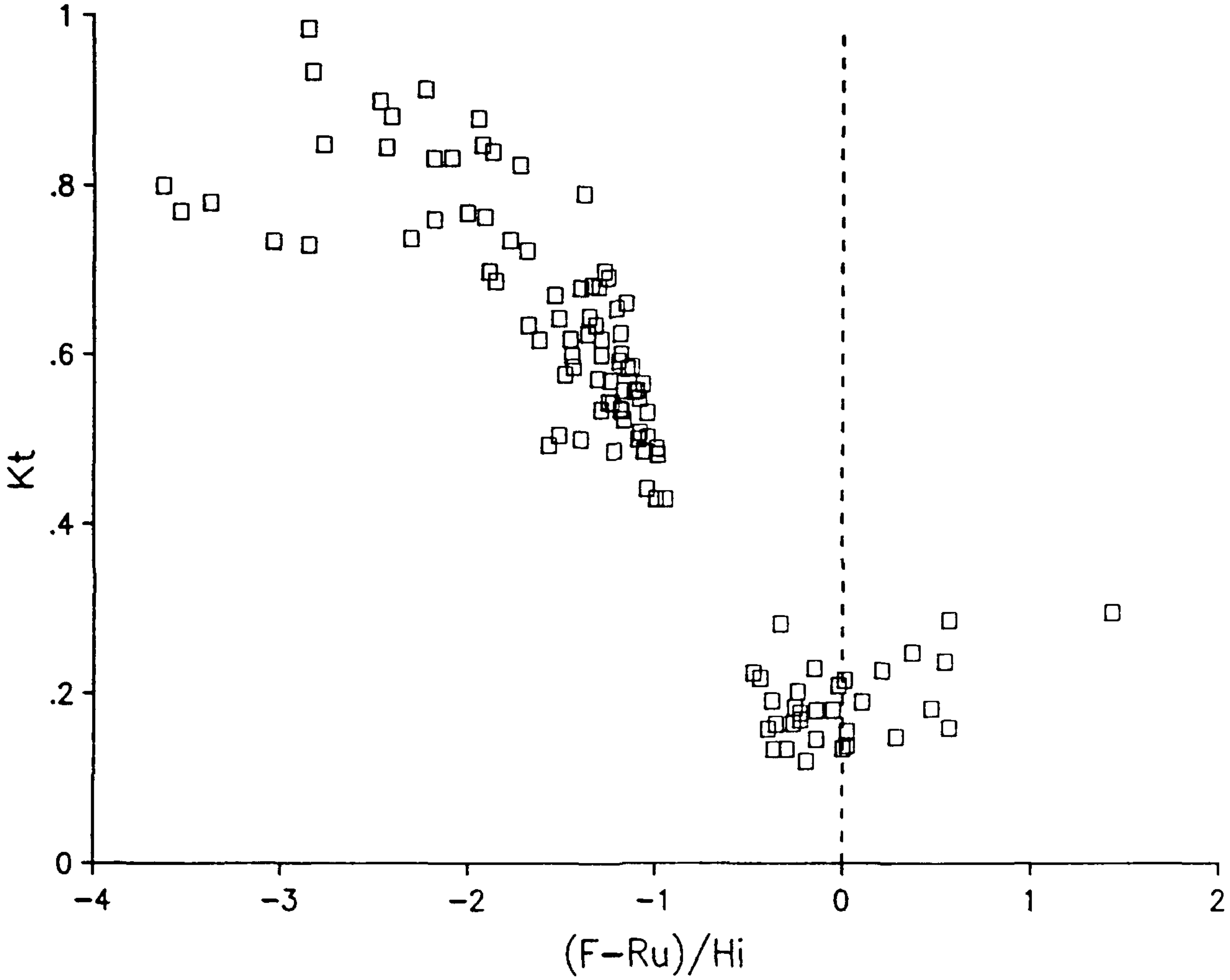


Figure 35. K_t versus $(F-Ru)/H_i$ for rubble breakwater

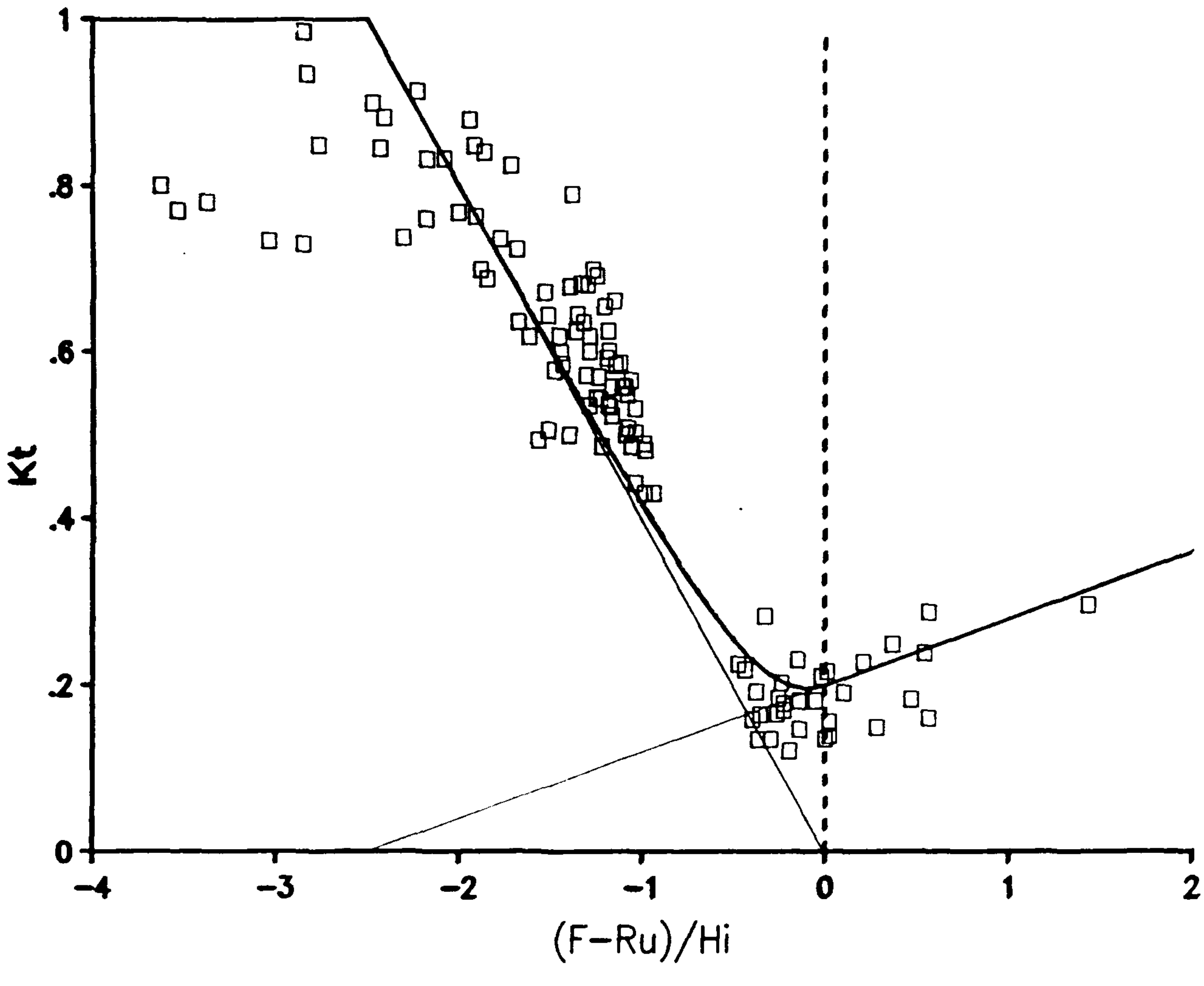


Figure 36. Comparison of proposed empirical equations to data for rubble breakwater

mathematically generated by adding the squares of the first two curves together and then taking their square root.

In conclusion, the proposed curves follow the data collected. For example, while the first curve, the curve responsible for transmission overtop, is equal to one, the second curve, the curve responsible for transmission through the structure, is equal to zero, as would be expected. In addition, both curves tend to the proper extremes. In other words, the first curve is equal to zero when the freeboard equals the runup, as was seen with the solid breakwaters, while the second curve trends towards one at an infinitely large value of the parameter, as was discussed earlier using the tidal example. In conclusion, both curves utilize the data collected in that the slopes of both lines were determined from these data.

COMPARISON OF SOLID AND RUBBLE TESTS

The goal of performing both solid and rubble tests was to compare these tests in the end and determine where transmission by overtopping controlled and where transmission through the structure controlled. Figure 37 illustrates K_t as a function of $(F\text{-}Ru)/H_i$, where Ru is determined by equation (8), for both permeable and impermeable breakwaters. Interestingly, Figure 37 resembles Figure 5, as was proposed by Ahrens (1987b) to distinguish between flow through and

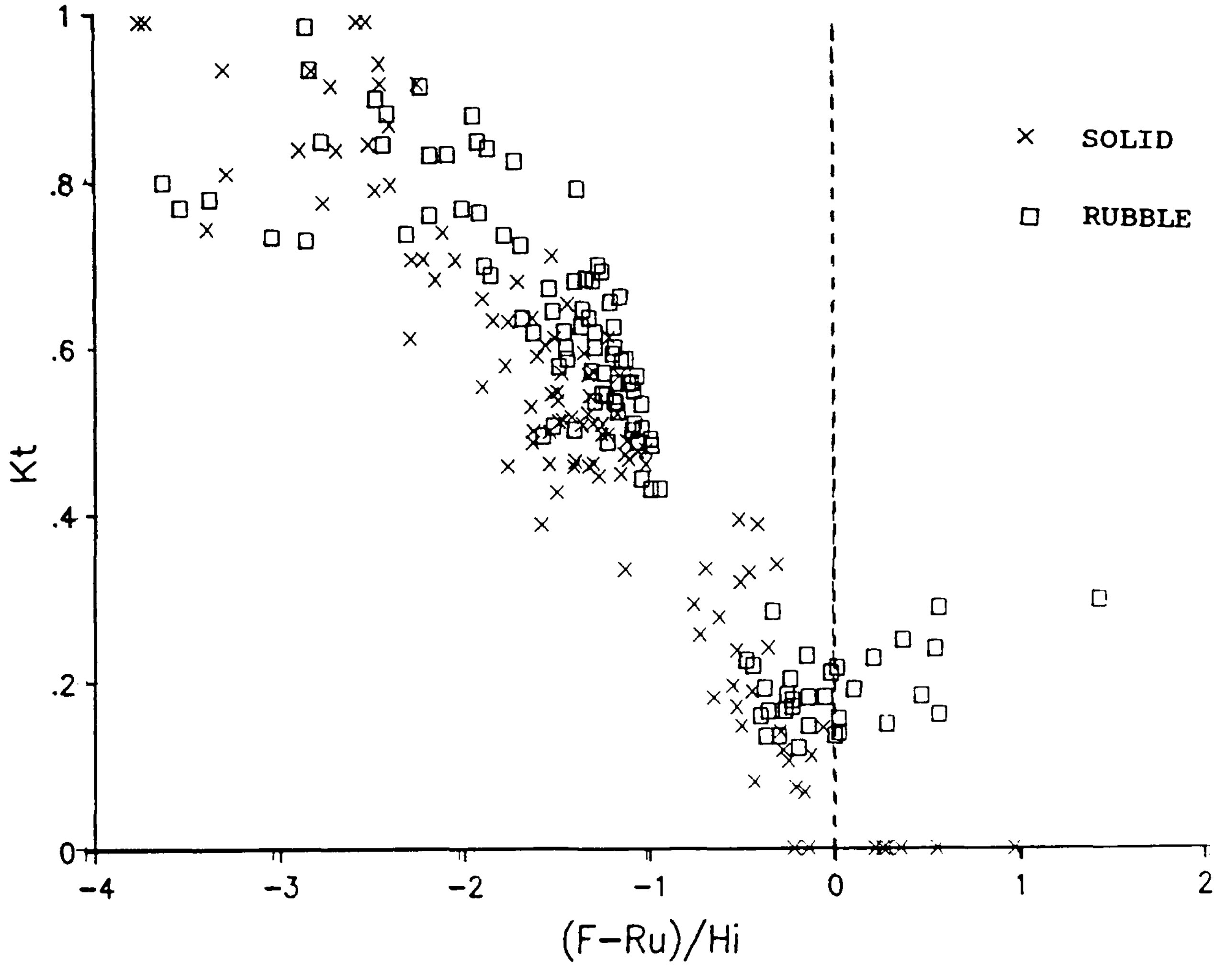


Figure 37. K_t versus $(F\text{-}Ru)/H_i$ for rubble and solid breakwater

overtop of the structure. As can be seen, rubble and solid breakwaters follow the same trend for $(F-Ru)/H_i$ values of approximately -0.4 and below, although K_t for rubble is generally greater. In this region, the waves hardly "feel" the breakwater and solid breakwaters are a good approximation of reef breakwaters when measuring transmission coefficients.

However, if one were to study a breakwater with a $(F-Ru)/H_i$ value of greater than approximately -0.4, the solid breakwater approximation would no longer hold, and K_t values are much larger than found for a solid breakwater. This is due to the fact that above the -0.4 value for $(F-Ru)/H_i$, the wave transmission is controlled by flow through the structure, and a solid breakwater is unable to account for this circumstance. In conclusion, solid breakwaters have been found to provide a lower-bound for K_t and results may be improved by incorporating a simple relationship to account for wave transmission through a rubble breakwater.

OTHER PARAMETERS

In addition to the parameters investigated and discussed above, several other parameters were considered in an effort to represent wave transmission by low-crested reef breakwaters. One of these parameters was $(F-H_i)/h_s$ as shown in Figure 38. At first glance, this parameter does not appear to represent trends in the data very well and, instead,

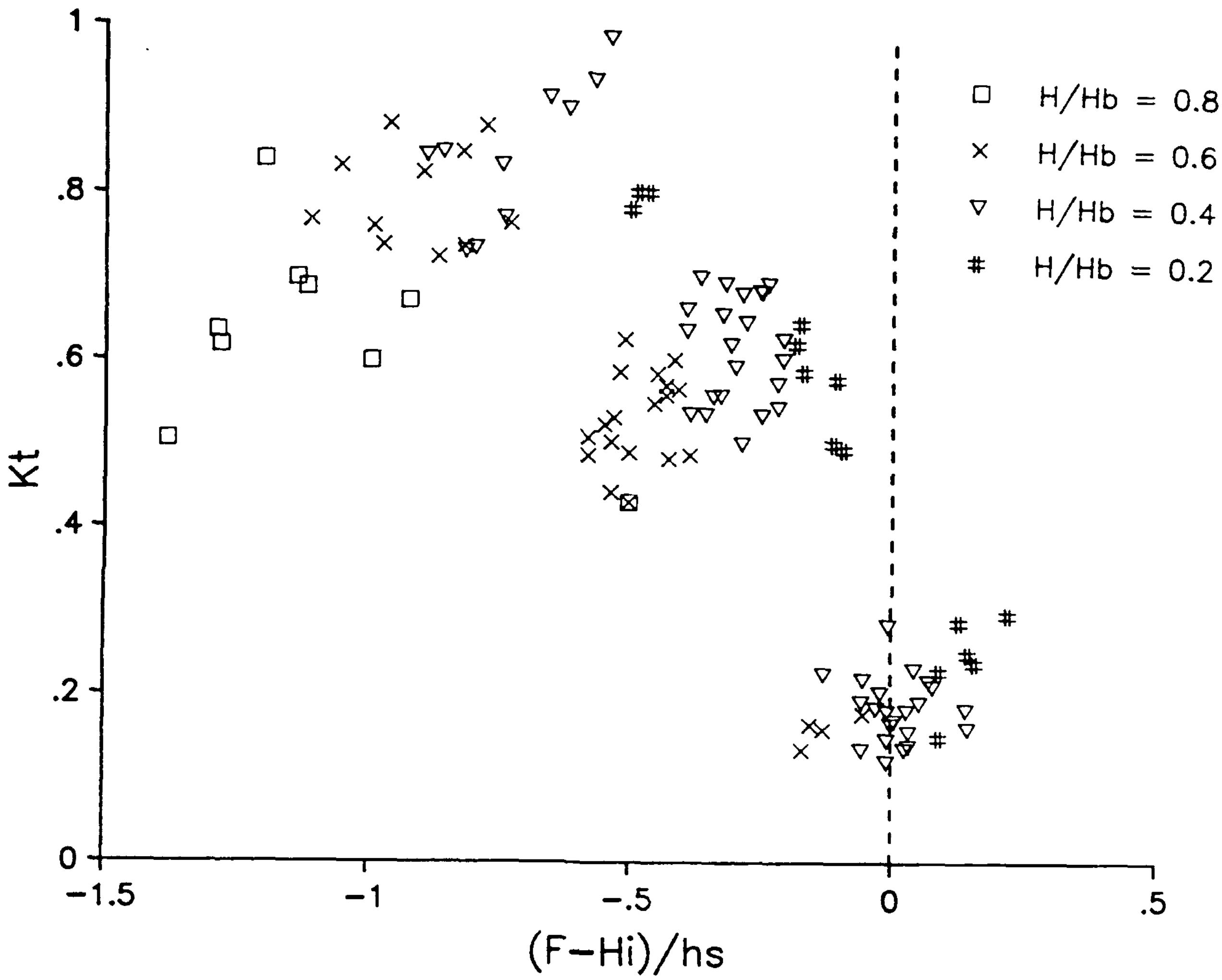


Figure 38. K_t versus $(F-H_i)/h_s$ illustrating its dependence on wave steepness

appears to introduce more scatter in the data. However, this is not the case. In Figure 38, the values of $(F-H_i)/h_s$ are separated into various categories of wave steepness. In this case, Seelig's wave steepness parameter was not used. Instead, the height of the wave in relationship to its breaking height was used to determine steepness. As summarized by McCormick (1973), the Miche formula gives the breaking wave height, H_b, as a function of local water depth and wavelength as:

$$H_b = L\frac{1}{7}\tanh(kh) \qquad \text{Eqn (9)}$$

where: H_b = breaking wave height
L = local wavelength
k = wave number
h = water depth

Based on this relationship, wave steepness is now defined by the ratio of the actual incident wave height H, to the theoretical breaking height, H_b. Figure 38 demonstrates that the H/H_b ratio is indeed a good indicator of the steepness of the wave. By grouping wave conditions into such categories of equal H/H_b ratios, it may be seen that the plot of K_t as a function of $(F-H_i)/h_s$ has some order after all. For very linear waves, or waves of almost perfect sinusoidal form, when H/H_b is about 0.2, K_t values are fairly large at any value of $(F-H_i)/h_s$. As waves are closer to breaking however, K_t decreases in a predictable way until, when H/H_b is about 0.8, transmission coefficients are smallest at any $(F-H_i)/h_s$ value. As a result, a parameter such as $(F-H_i)/h_s$, which does not itself include any dependence on wave steepness, may in fact be used to represent the data trends if the data are first grouped into various categories of wave steepness.

An additional parameter investigated was the so-called reef transmission variable, P, as proposed by Ahrens (1987b). This variable is composed of the bulk number and the wave steepness, where $P = (Bn)(H_i/L)$. The bulk number, Bn, is defined by Ahrens (1987b) as:

$$Bn = \frac{A_t}{d_{50}^2} \qquad \text{Eqn (10)}$$

where: A_t = area of the breakwater cross section
d_{50} = median diameter of armor stone

and it represents the number of stones in the cross-section. Shepard and Hearn (1989) proposed the following empirical equation relating K_t to the reef transmission variable for values of relative freeboard greater than one:

$$K_t = \frac{1}{1+P^{0.59}} \qquad \text{Eqn (11)}$$

Figure 39 shows equation (11) plotted for the data gathered in this study for values of F/H_i greater than one. As can be seen, the empirical equation does not do a very good job in predicting K_t values. This method overpredicts over the whole range of conditions studied. The reason behind this overprediction may be due to the fact that Shepard and Hearn (1989) proposed this equation using the data gathered by

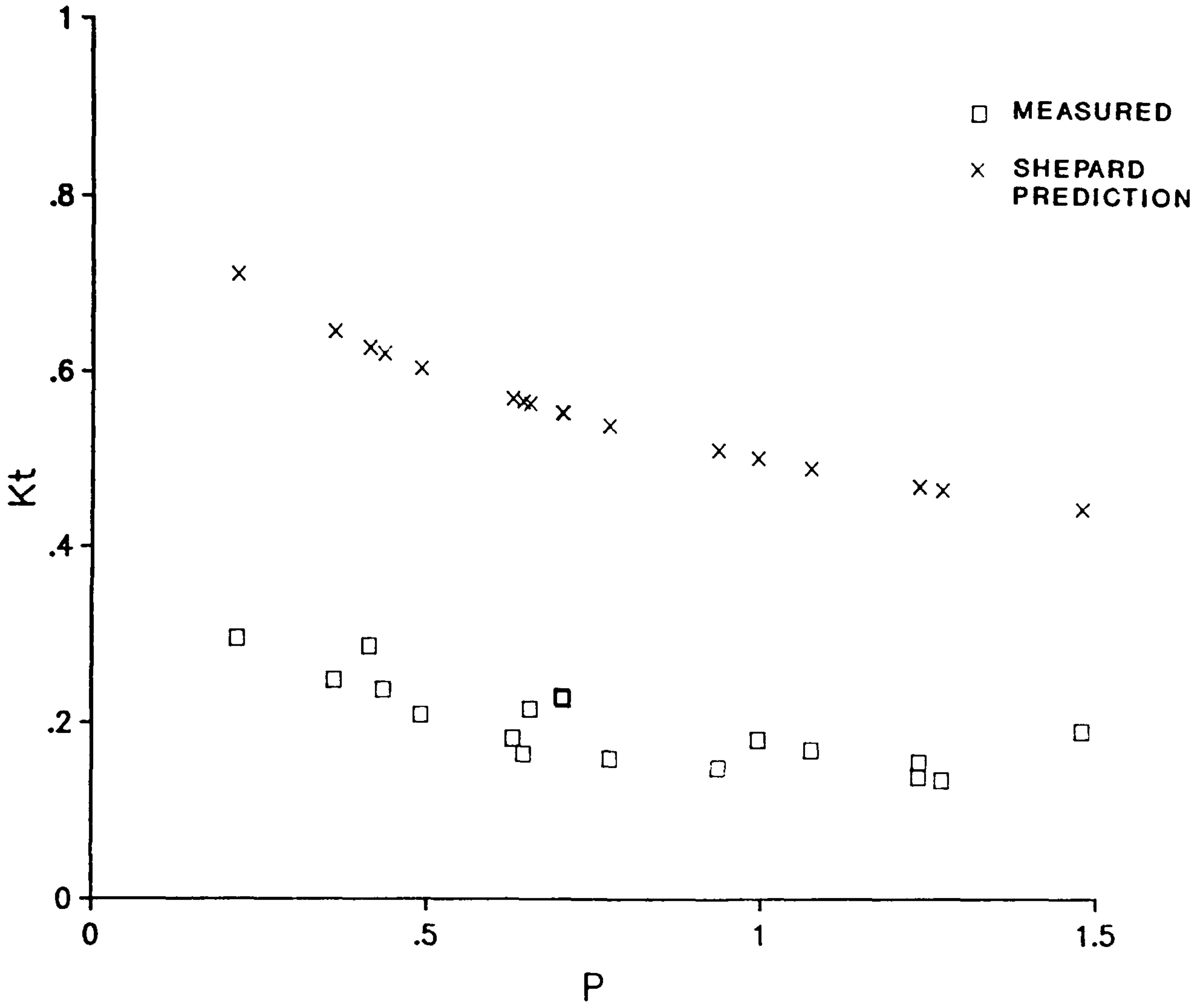


Figure 39. Comparison of Shepard (1989) equation to data

Ahrens (1987b), which was primarily concerned with the stability of reef breakwaters. Due to this concern with stability, Ahrens used much smaller stone sizes, and thus larger bulk numbers than were used in this study. In fact, bulk numbers tested by Ahrens were in the range of 200 to 600 and were an order of magnitude larger than those used in this study, which ranged from 12 to 50. In addition, Ahrens tested some waves with very small wave steepness, in some cases an order of magnitude smaller than those used in this study. As a result, values of P tested by Ahrens are in the same range of those used in this study, but wave transmission characteristics are dramatically different.

In conclusion, there does seem to be some dependence on stone size as far as the transmission characteristics of a reef breakwater are concerned, but more study is needed before any design criteria may be generated from it. More importantly, it appears that the reef transmission parameter, P, is not as effective at discriminating between the transmission characteristics of reef breakwaters as was first thought.

RUBBLE BREAKWATER--IRREGULAR WAVES

The next portion of the study was the investigation of irregular wave transmission past reef breakwaters. In this part, rubble breakwater models were subjected to a random sea.

The crest heights and water depths tested were the same as in the regular wave section of the study described earlier. Due to the fact, however, that this part of the study dealt with irregular waves, there was no longer a constant height and period associated with each incident wave. Thus, it was necessary to use a new method to measure and analyze these random waves.

The zero-upcrossing method, as shown in Figure 40, was chosen for analyzing random waves in order to obtain values of height and period. This method involves identifying individual wave heights and periods between the up-crossings of the wave record about the still water level. Several computer programs were written in True Basic to perform the zero up-crossing analysis on the wave data. Once all of the individual heights and periods were known for a wave record, a statistical analysis was performed in order to find several descriptive parameters of the random sea. This included the significant wave height, H_s, defined as the average height of the 1/3 highest waves measured, as well as the root mean square height of the waves, H_{rms}, which is defined as the square root of the average squared-wave heights in the sea state.

This statistical method was at first utilized to measure both incident and transmitted waves in the random sea. However, a problem arises with this method since it cannot account for the effect of reflected waves which are also

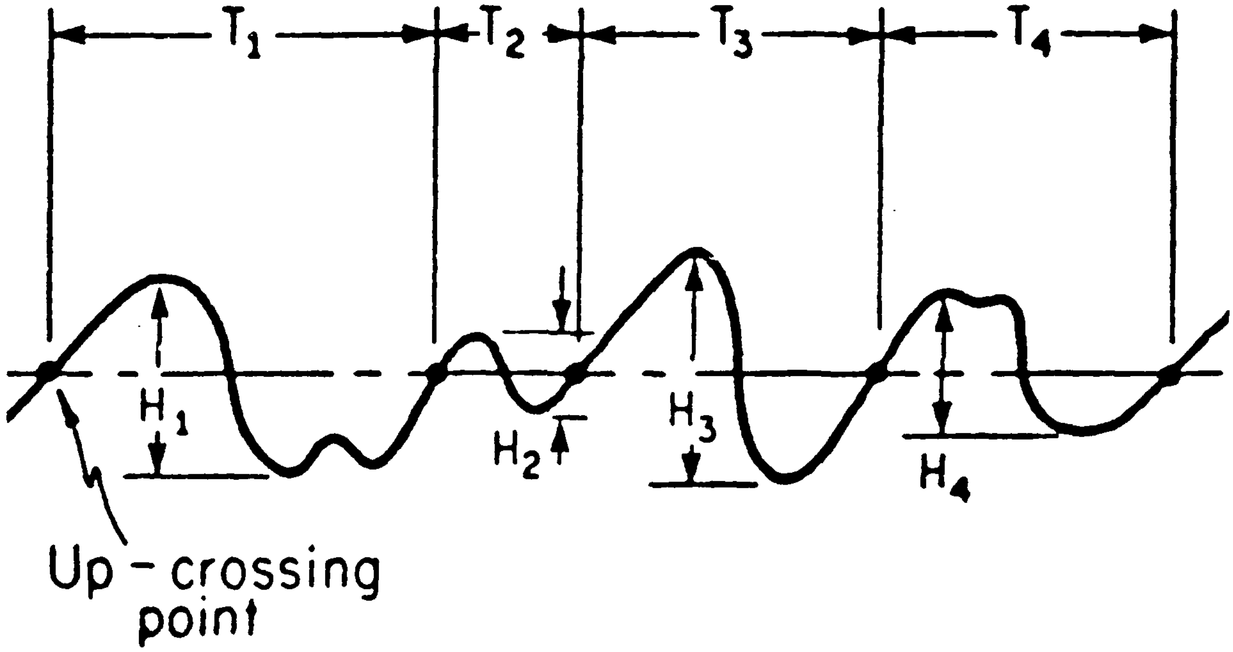


Figure 40. Zero-upcrossing method

present in the random sea. Due to the fact that the incident waves are now irregular, there no longer exists a clearly defined wave envelope in front of the breakwater, as there did with regular waves. Thus, the method of Goda and Suzuki (1976) was adopted for measuring both the incident and reflected waves in front of the breakwater as well as the transmitted and reflected waves in the lee of the breakwater.

The method of Goda and Suzuki (1976) uses a pair of wave gages in front of the structure and a pair of gages behind the structure, as shown in Figure 41, along with spectral analysis methods to analyze incident and reflected waves. The two gages in the pair are meant to be very close together, in this study they are 6 inches apart, and they are kept stationary during the data acquisition process. Following the acquisition of the data by the computer-based data acquisition system, a Fast Fourier Transform is run on the data. This Fast Fourier Transform (FFT) yields the Fourier coefficients A and B, associated with cosine and sine components at each frequency in the random sea. In this case, however, both A and B contain the effects of the incident and reflected waves. With these coefficients, Goda and Suzuki (1976) then propose the equations shown in Figure 42 to solve for a_I and a_R, which are the incident and reflected wave amplitudes for each frequency. As a result, two separate wave amplitude spectra are obtained, one for the incident waves, a_I and one for the reflected waves a_R.

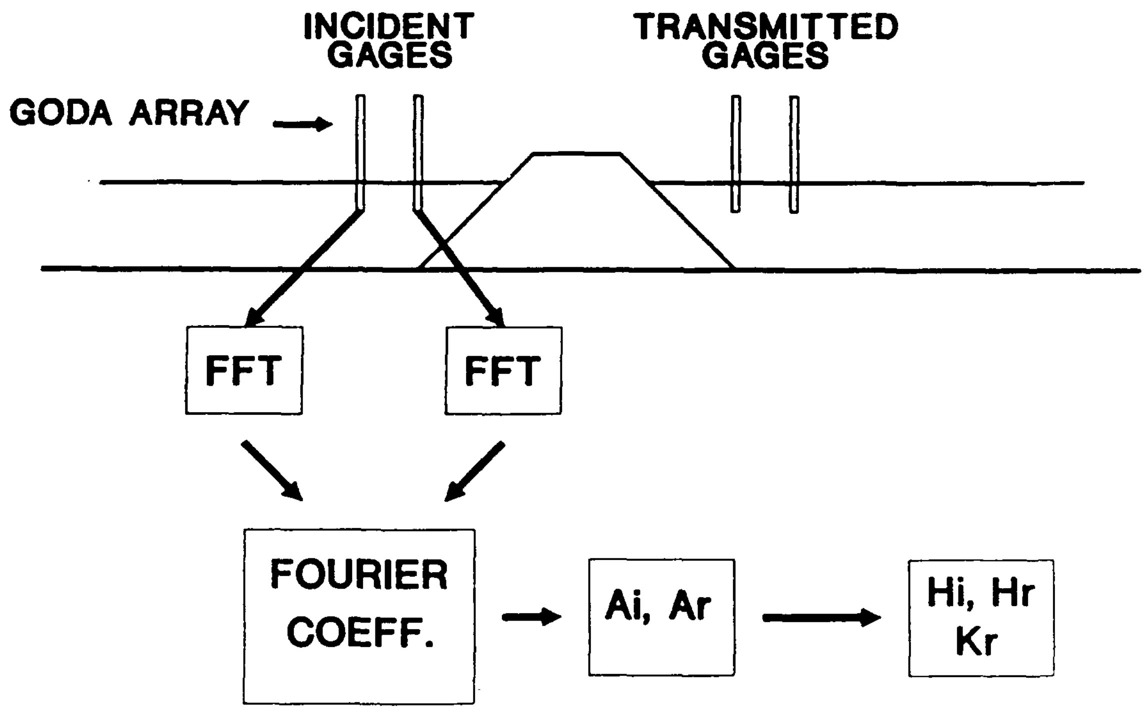
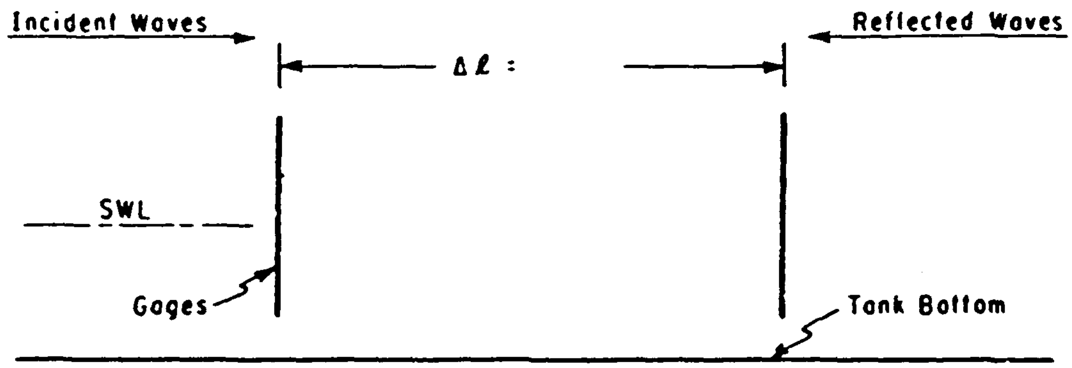


Figure 41. Goda array setup for analysis of irregular waves



$$a_I = \frac{1}{2|\sin k\Delta\ell|}\sqrt{(A_2 - A_1 \cos k\Delta\ell - B_1 \sin k\Delta\ell)^2 + (B_2 + A_1 \sin k\Delta\ell - B_1 \cos k\Delta\ell)^2}$$

$$a_R = \frac{1}{2|\sin k\Delta\ell|}\sqrt{(A_2 - A_1 \cos k\Delta\ell + B_1 \sin k\Delta\ell)^2 + (B_2 - A_1 \sin k\Delta\ell - B_1 \cos k\Delta\ell)^2}$$

where

A,B = spectral coefficients

k = wave number = $\frac{2\pi}{L}$

$\Delta\ell$ = gage spacing

$$0.05 \le \frac{\Delta\ell}{L} \le 0.45$$

Figure 42. Goda and Suzuki (1976) method of irregular wave analysis from Seelig (1980)

Following the Goda and Suzuki (1976) method to determine the values of a_I and a_R, statistical analysis is then used to determine the significant incident, transmitted, or reflected wave heights in front of the breakwater. A similar procedure was used behind the breakwater to determine the significant transmitted and reflected wave heights where reflection is now due to the energy absorbing beach located at the end of the test channel. As a result, wave transmission coefficients for the irregular wave tests are then defined as the ratio of the transmitted significant wave height to the incident significant wave height.

Following the reduction of all of the irregular wave data by the above method, values of K_t were plotted against the same parameters that were used for the regular wave section of the study. This time however, the regular wave data and the irregular wave data are plotted on the same graph in an attempt to compare and contrast the transmission characteristics of reef breakwaters in random seas as opposed to regular seas.

The first parameter, relative freeboard, F/H_i, is shown in Figure 43 for both regular and irregular waves, where H_i is the significant incident wave height in an irregular sea. What is interesting, is that the irregular wave values follow the same trend as did the regular wave values. In addition, the irregular wave values correlate well with the regular wave values very well. Several other parameters were also

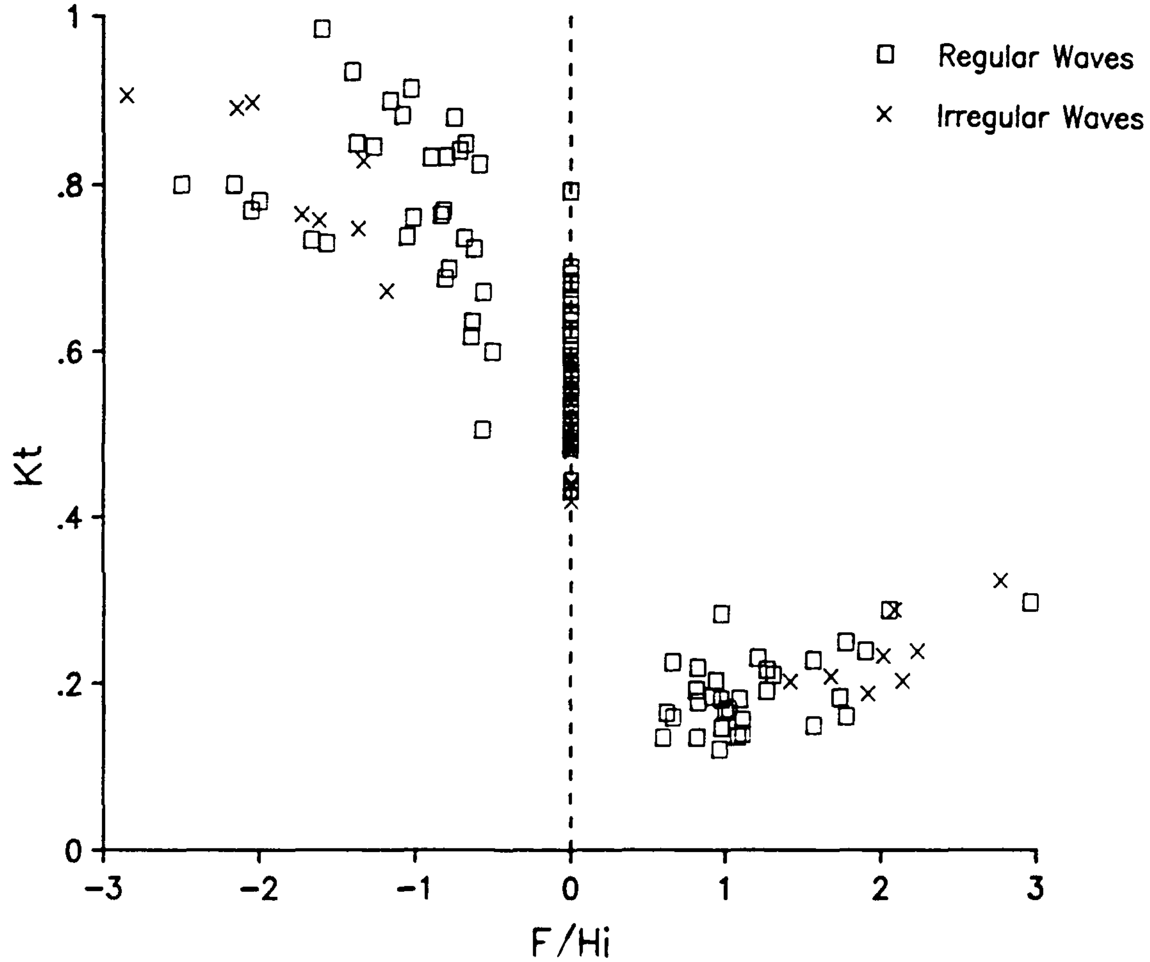


Figure 43. K_t versus relative freeboard for irregular and regular waves

investigated, including the F/Ru and the $(F-Ru)/H_i$ parameters where, once again, significant wave height was used to represent random seas. K_t is shown as a function of both F/Ru and $(F-Ru)/H_i$ in Figures 44 and 45 for both regular and random waves. Again, as was the case for relative freeboard, these parameters show very little difference in trends between the values of K_t gathered for regular waves and those gathered for irregular waves.

With all three parameters, the values of irregular and regular waves correlate with each other remarkably well. This leads to an important conclusion of importance to the design engineer: irregular waves can be modeled in the laboratory by regular waves with the peak frequency and significant height of the irregular wave, for the study of transmission characteristics of reef breakwaters. This is of importance to the design engineer because it may not be necessary to spend a great deal of time and money programming, generating, and carrying out an irregular wave study of a reef breakwater project. Instead, a cheaper and much quicker regular wave study could be carried out for the purpose of investigating transmission characteristics of reef breakwaters. However, it must be understood that this approximation of irregular waves with regular waves of the same peak frequency and significant height should be used only for transmission studies of reef breakwaters; breakwater stability and shoreline erosion studies would require irregular wave tests.

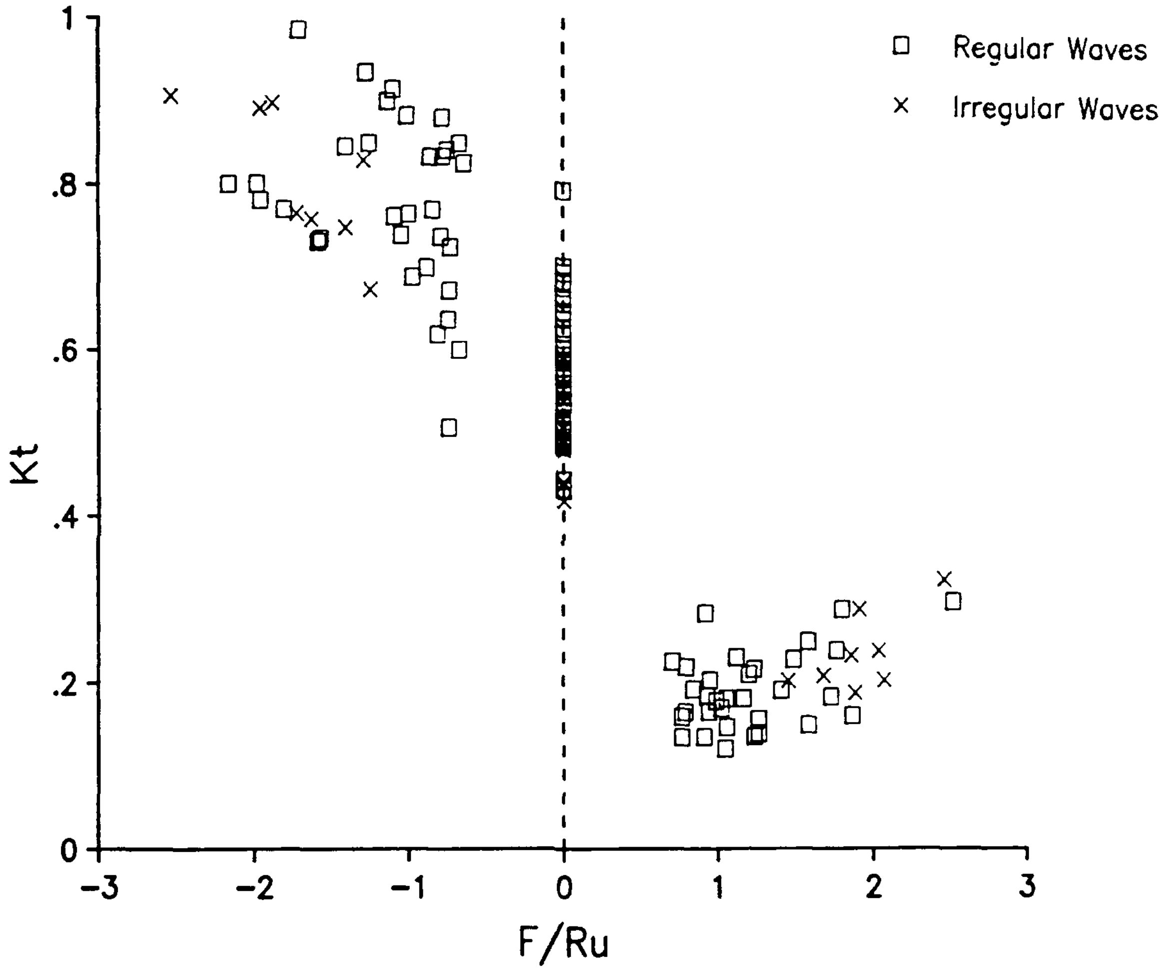


Figure 44. K_t versus F/Ru for irregular and regular waves

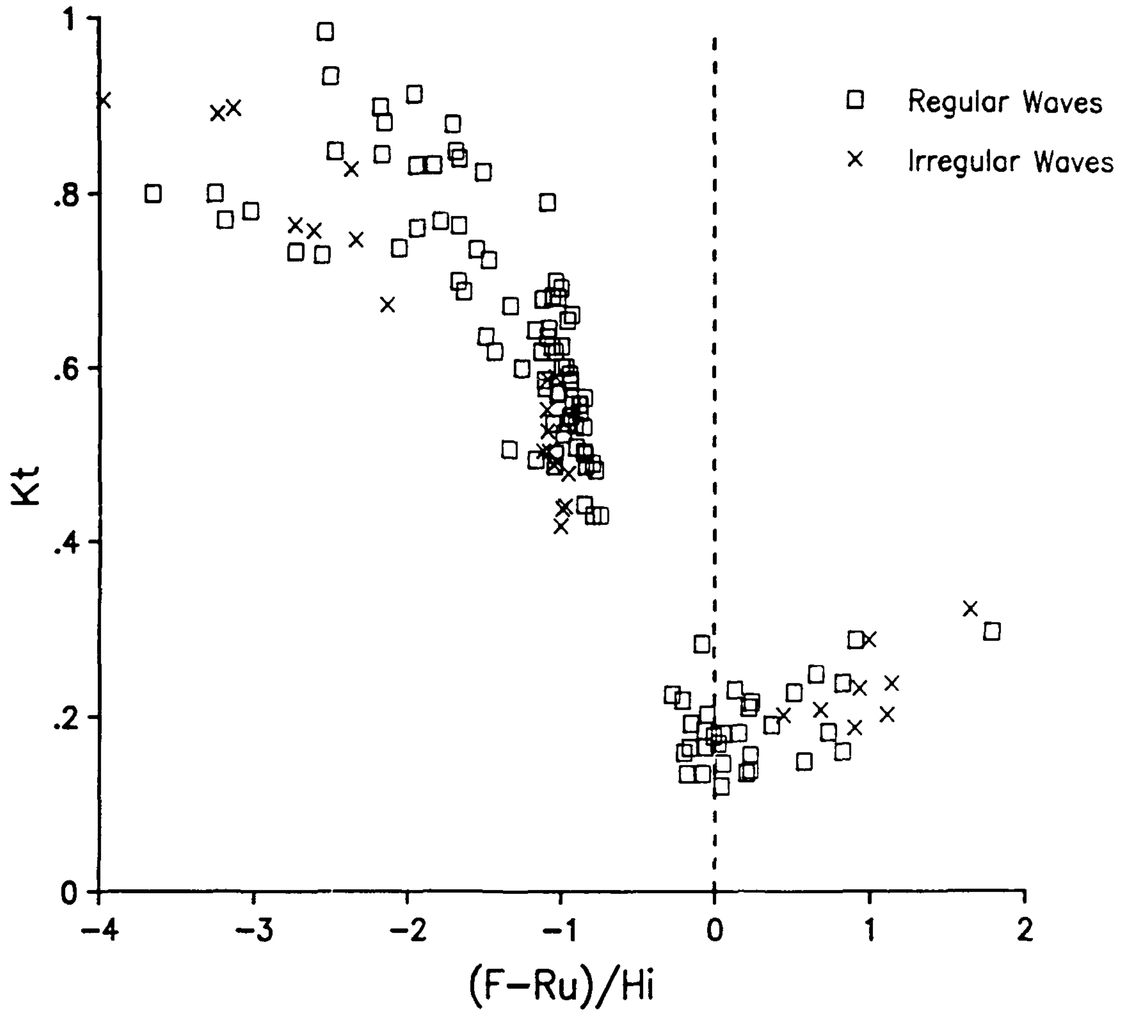


Figure 45. K_t versus $(F\text{-}Ru)/H_i$ for irregular and regular waves

HARMONIC PROPAGATION TESTS FOR REGULAR WAVES

The harmonic propagation portion of this study is a result of a suggestion by Professor Robert Dalrymple of The University of Delaware. It was the hypothesis of Professor Dalrymple that as regular waves propagated over a low-crested breakwaters, their harmonics, or higher multiples of the wave's fundamental frequency, were "released", and once released, these harmonic waves would propagate according to linear wave theory. This release of harmonics would cause a spatially-varying transmitted wave behind the breakwater as was being found. These variations in wave form were studied over the length of the test channel through the use of a sonic wave gage. An example of the output from one of these studies is shown in Figure 46. As can be seen, there is a difference in the wave form of the incident waves as compared to the transmitted waves. Behind the breakwater, transmitted were modulated as they would be if several wave frequencies were present. Thus, it was concluded, based on several other similar studies, to investigate this harmonic propagation hypothesis.

For these tests, three wave gages were used behind the breakwater, and all three gages were again interfaced with the computer-based data acquisition system, as was the case for the irregular wave tests. All three gages were used to measure transmitted height and were held stationary during the

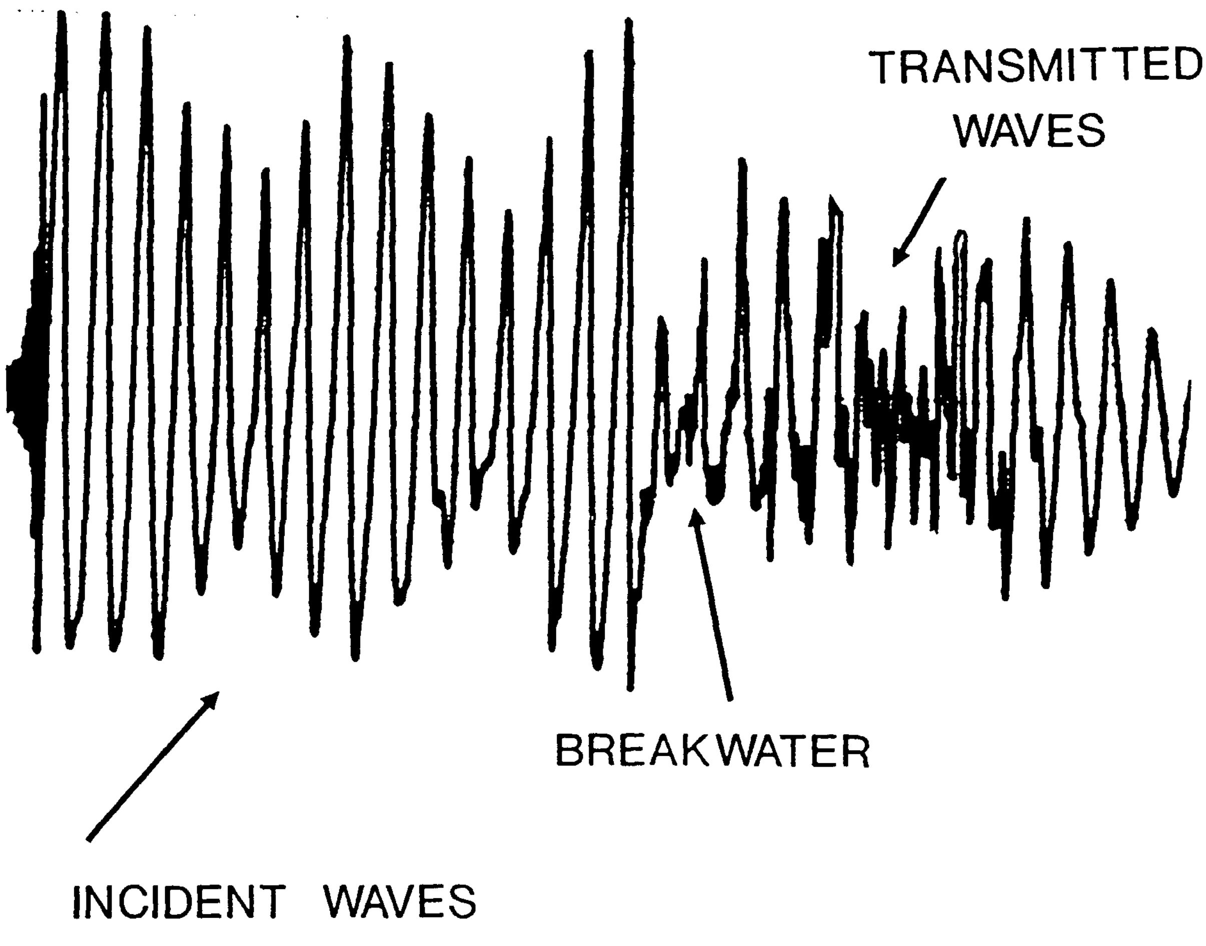


Figure 46. Sonic wave gage test showing variations in wave height along the channel

testing. Following the data collection process, the data were again analyzed by an FFT routine. However, this time, unlike the irregular wave studies, only the values for A and B at the fundamental frequency and each harmonic out to the fourth were of interest. With these spectral coefficients, it was now possible, using linear wave theory, to predict the wave form at any position in the test channel behind the breakwater as $n(x,t) = A_0 + A_1 \cos(kx-wt) + B_1 \sin(kx-wt) + A_2 \cos(k_2x-2wt) + B_2 \sin(k_2x-2wt) + \ldots$ where k_2 is the linear wave number associated with the second harmonic propagating at a frequency of 2w and so forth. Thus, if the incident waves did in reality release their harmonics as they passed over the breakwater, and these harmonics traveled according to linear wave theory, it should be possible to measure the wave at one location in the channel behind the breakwater, analyze the spectral coefficients, reform the wave using linear wave theory, and then predict the wave at another location in the channel.

This hypothesis was tested using the three wave gages placed behind the breakwater. The gage closest to the breakwater was used as the analysis gage. From this gage, all of the spectral coefficients were obtained. Next, these coefficients were reformed into linear waves and theoretically propagated down the channel to the location of the other two wave gages. The distance that the wave was theoretically propagated was either 18 or 24 inches. The simple schematic shown in Figure 47 demonstrates the process carried out in this portion of the study for two wave gages.

The results of these experiments appear in Figures 48-51. Figures 48 and 49 are for an 8 inch breakwater in 8 inches of water with waves having a frequency of 0.9 Hz. and a significant height of 3.4 inches. Figure 48 shows the recreated data propagated 18 inches down the channel, while Figure 49 shows the recreated data propagated 24 inches down the channel. With both figures, the recreated data predicts the actual data well in the crests but not in the troughs. What is especially interesting is the difference in the wave forms between two points in the channel which are only 6 inches apart. There is a difference in not only wave height, but in presence of additional crests as well. Figures 50 and 51 illustrate the same comparison, but in these cases, the breakwater is 6 inches in 8 inches of water with a wave frequency of 1.1 Hz. and a significant height of 1.3 inches. Again, there is a difference in the wave forms between the two stations investigated as in wave height and additional harmonics, but the recreated data does a good job in predicting the actual data considering that there was not a correction made in the predicted data for reflection off of the beach and the breakwater.

In conclusion, it may be stated that as incident regular waves pass over a breakwater, they release their harmonics which propagate as linear waves behind the breakwater.

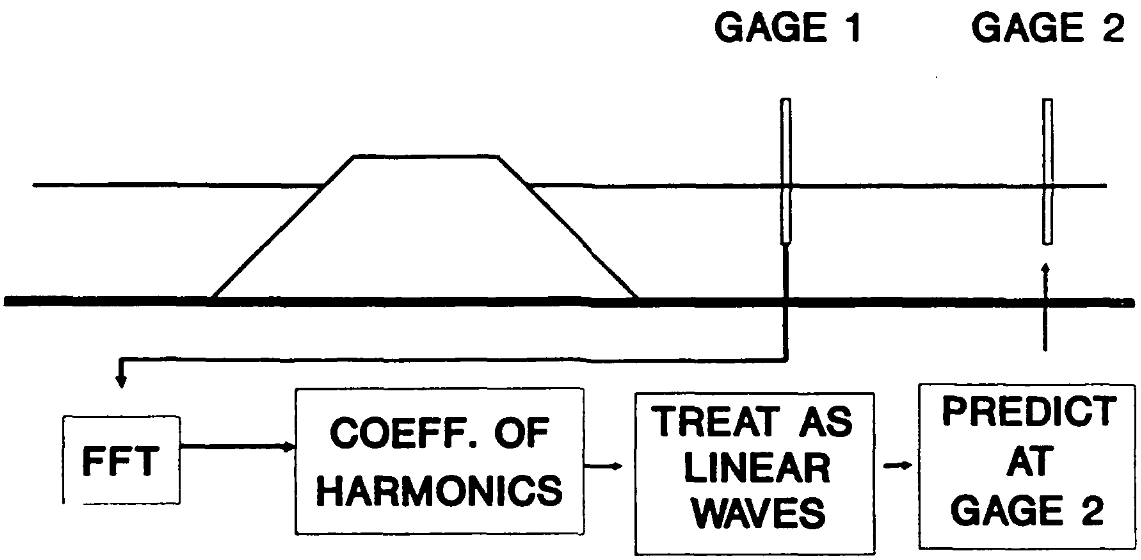


Figure 47. Setup for harmonic propagation experiments

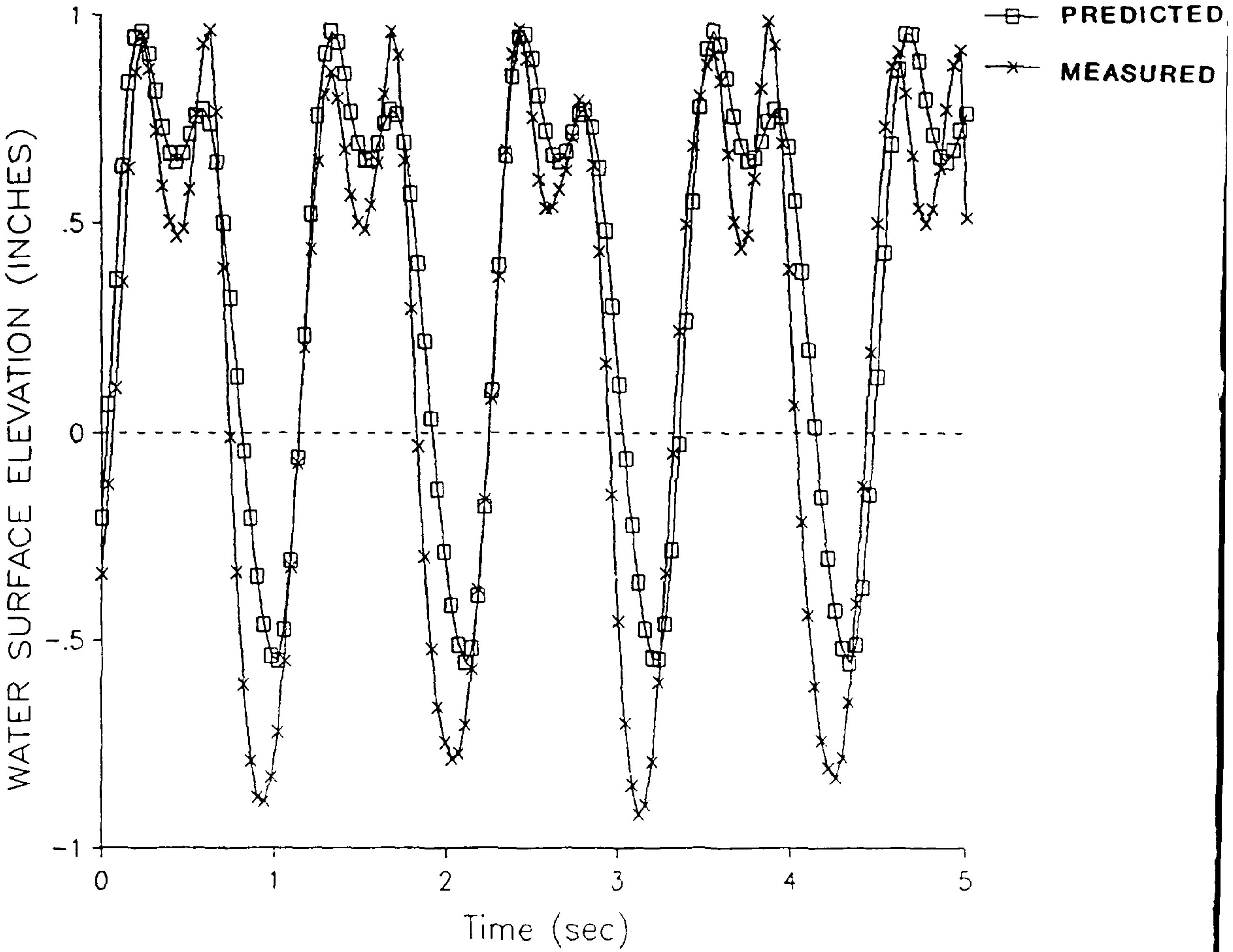


Figure 48. Comparison of measured to predicted data for harmonic propagation tests (18 inch projection)

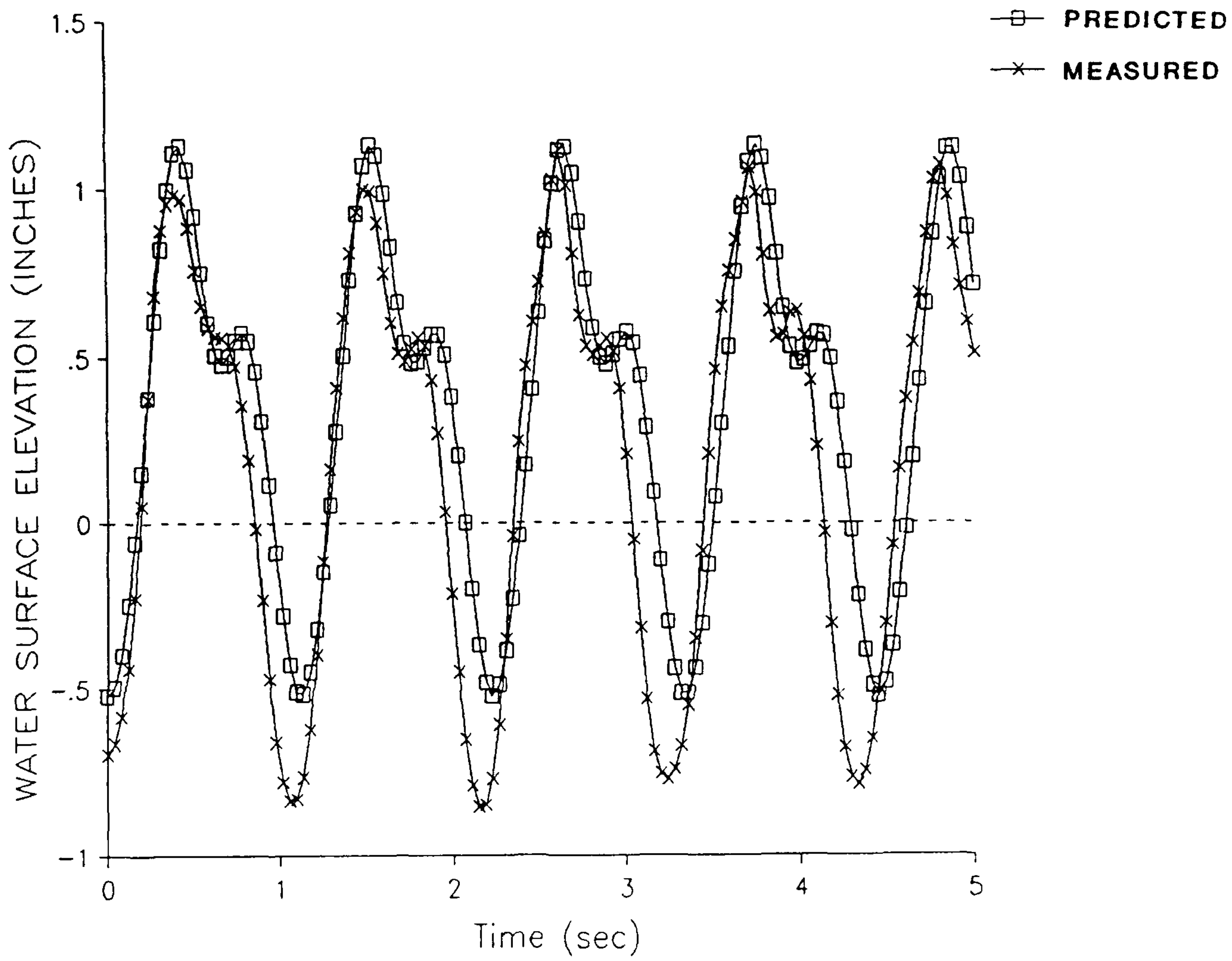


Figure 49. Comparison of measured to predicted data for harmonic propagation tests (24 inch projection)

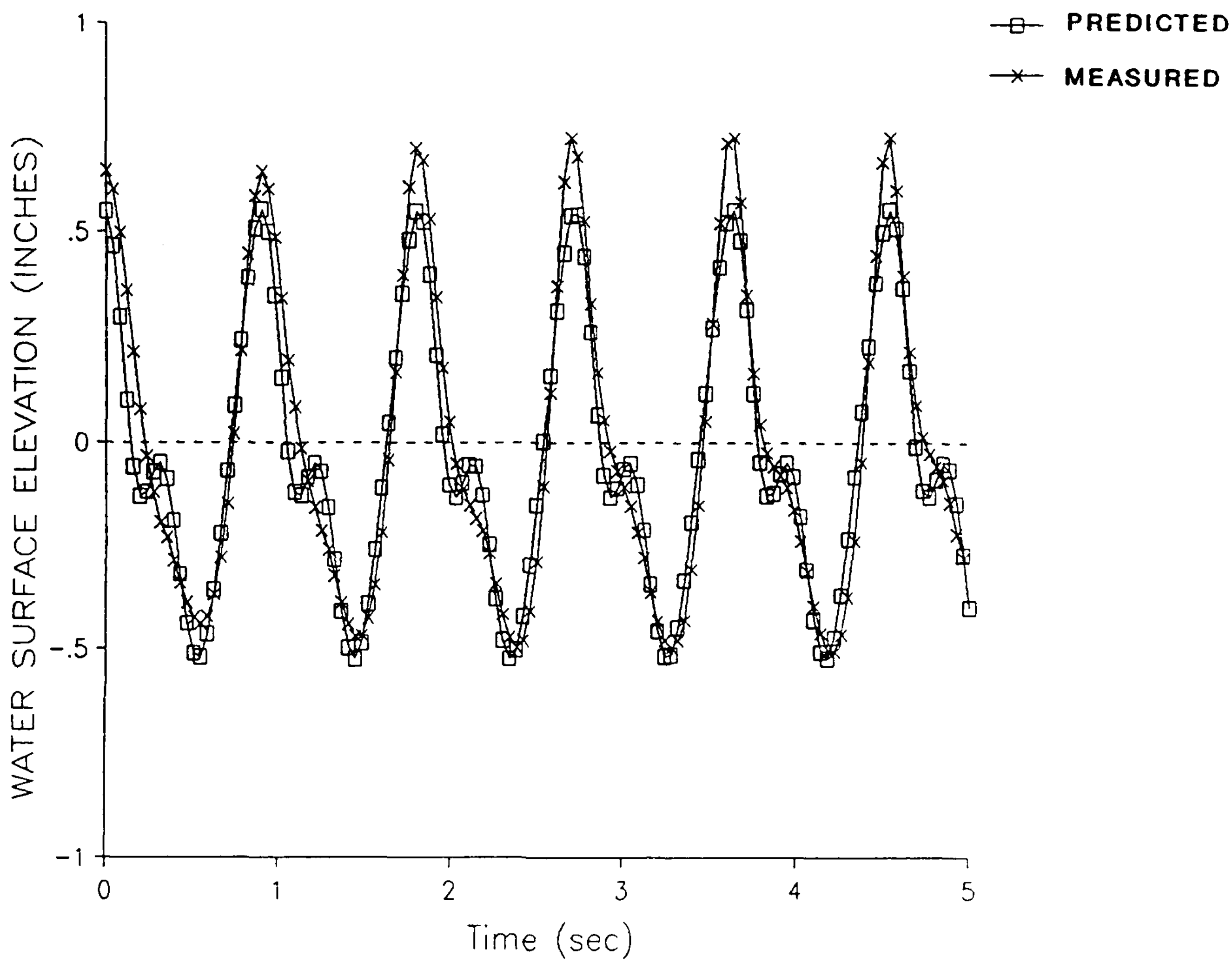


Figure 50. Comparison of measured to predicted data for harmonic propagation tests (18 inch projection)

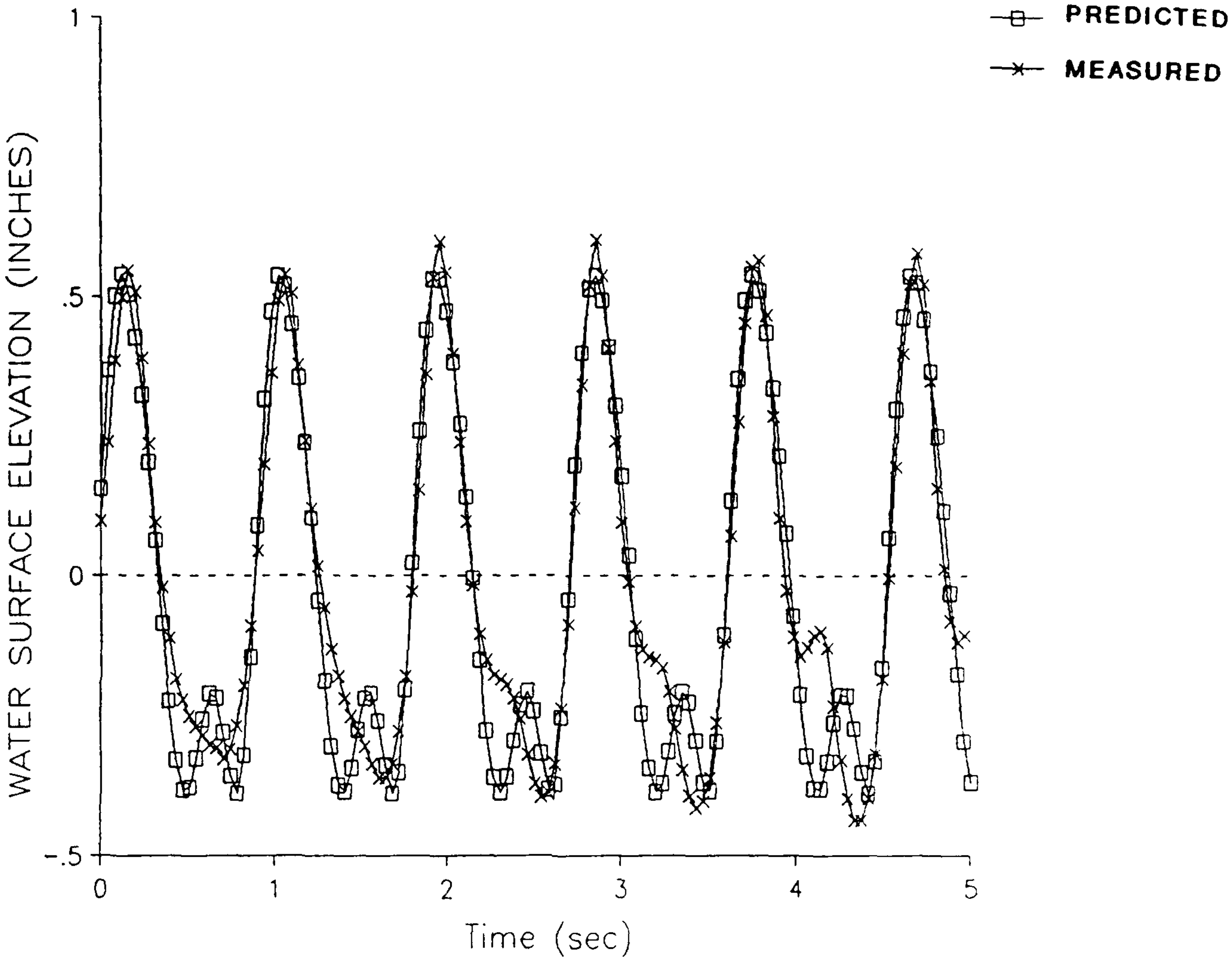


Figure 51. Comparison of measured to predicted data for harmonic propagation tests (24 inch projection)

However, this may not always hold true, especially in shallow water conditions with long period waves. In shallow water, this hypothesis is not as accurate because linear wave theory does not really apply very well. It may be necessary to describe the waves using cnoidal wave theory instead, which is a highly nonlinear wave theory described by the Shore Protection Manual (1984). Overall, however this hypothesis deserves much further study since it may be possible to predict the spatially varying wave height behind a breakwater using this theory if the incident wave height is known. This capability combined with the new parameters developed earlier in the study, and current diffraction analysis may one day enable an engineer to predict more accurately the wave height at any location behind a breakwater, especially in areas near the shoreline.

CONCLUSION

Based on the material presented on reef breakwaters in this study, the following conclusions may be drawn:

* Reef breakwater design is a difficult problem. Aside from the fact that there are numerous variables which control the wave transmission through, overtop, and around these structures, the structures themselves have a very complex effect on the site at which they are located which is very difficult to predict and evaluate.

* The process of wave transmission not only varies with the physical features of the breakwater such as freeboard, but also with the aspects of the incident wave such as wave steepness. Thus, at times it is very difficult to separate the two in order to decide which is the dominant aspect in the case under study.

* There are a number of variables which control the transmission characteristics of a reef breakwater. The main variables controlling K_t are freeboard, wave runup, and incident wave height and period. Combinations of these parameters develop trends in the data which may benefit a design engineer in understanding reef breakwaters. All parameters are also relatively simple and easy to use.

* The $(F-Ru)/H_i$ parameter describes the transmission characteristics of reef breakwaters better than any other parameter considered. This is due to the fact that this

parameter does not fail to discriminate among breakwaters with a zero freeboard under various water conditions. Thus, this is the most practical parameter available from an engineering standpoint. With this parameter, an engineer who wished to design and build a reef breakwater with a zero freeboard would be able to predict the value of K_t for that breakwater with more confidence than he/she would be able to using a parameter such as the relative freeboard.

* A solid breakwater model will approximate a rubble model reasonably well for values of $(F-Ru)/H_i < -0.4$. Above this value, transmission through the structure begins to dominate and a solid approximation is no longer valid. Thus, if an engineer desired to build a very low-crested breakwater, with a constant negative freeboard, it would be possible to model that structure in the laboratory with a solid breakwater of similar geometry.

* Irregular wave results exhibit a similar trend and "over-lay" regular wave results when significant wave height is used to characterize the random waves. Thus, an engineer would be able to perform wave transmission studies of a reef breakwater in the laboratory using regular waves, and expect to find similar results in the field under irregular wave conditions, if the regular waves generated in the laboratory were of the same peak frequency and significant heights as those waves found at the site.

* Wave propagation behind a reef breakwater is a complicated process. However, as a first approximation of this process, the wave field in regular waves behind a breakwater may be characterized by superposition of regular linear waves, with frequencies corresponding to the harmonics of the incident wave frequency

* Due to the fact that the study of reef breakwaters is a complicated process, there is a great deal of additional research which must be done. First, the spatial variation in wave heights behind a breakwater should be further investigated with the possible use of additional non-linear wave theories. Second, three-dimensional wave transmission should be investigated in hopes of determining how transmission overtop, transmission through the structure, and diffraction interact behind a breakwater. Third, additional research needs to be conducted in the field of sediment transport behind breakwaters as a function of the wave transmission characteristics.

ACKNOWLEDGMENTS

I would like to thank my advisor, Dr. David L. Kriebel, of the Naval Academy Ocean Engineering Program, Ms. Louise Wallendorf of the U.S. Naval Academy Hydromechanics Laboratory, Mr. John R. Hill, Branch Head, U.S. Naval Academy Hydromechanics Laboratory, the remainder of the Hydromechanics Laboratory staff, and the many others who helped out so generously and gave up so much of their time during the past year and a half that it took to complete this project. Last but not least, I would like to thank my parents, Harry and Sandra Davies, for all their support, and especially my roommate, Jeff Uhde, for putting up with me this past year.

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APPENDIX A

SOLID BREAKWATER--REGULAR WAVE DATA

Solid Breakwater-Regular Waves

Hs (in.)	h (in.)	f (Hz.)	L (in.)	LO (in.)	Ht (in.)	Hi (in.)	Hr (in.)	Kt	Kr	Kd	F (in.)
4.0000	4.0000	1.10	32.77	50.79	.63	1.20	.30	.53	.25	.81	.00
4.0000	4.0000	1.10	32.77	50.[illegible]9	.90	1.80	.50	.50	.28	.82	.00
4.0000	4.0000	1.10	32.77	50.79	1.05	2.33	.53	45	.23	.86	.00
4.0000	4.0000	1.10	32.77	50.79	.85	2.53	.93	.34	.37	.87	.00
4.0000	4.0000	.90	41.25	75.88	.53	1.03	.33	52	.32	.80	.00
4.0000	4.0000	.90	41.25	75.88	.83	1.80	.55	.46	.31	.83	.00
4.0000	4.0000	.90	41.25	75.88	1.00	2.23	.68	.45	.30	.84	.00
4.0000	4.0000	.90	41.25	75.88	.93	2.00	.80	.47	.40	.79	.00
4.0000	4.0000	.70	54.27	125.43	.55	.93	.38	.59	.41	.69	.00
4.0000	4.0000	.70	54.27	125.43	.88	1.43	.38	.62	.26	.74	.00
4.0000	4.0000	.70	54.27	125.43	1.25	1.90	.40	.66	.21	.72	.00
4.0000	4.0000	.70	54.27	125.43	1.05	2.25	.65	.47	.29	.84	.00
4.0000	4.0000	.55	69.98	203.17	.35	.60	.20	.58	.33	.74	.00
4.0000	4.0000	.55	69.98	203.17	.60	.88	.33	.69	.37	.63	.00
4.0000	4.0000	.55	69.98	203.17	.85	1.33	.38	.64	.28	.71	.00
4.0000	4.0000	.55	69.98	203.17	1.18	2.15	.65	.55	.30	.78	.00
6.0000	6.0000	1.10	38.33	50.79	.80	1.60	.50	.50	.31	.81	.00
6.0000	6.0000	1.10	38.33	50.79	1.20	2.53	.48	.48	.19	.86	.00
6.0000	6.0000	1.10	38.33	50.79	1.55	3.24	.51	.48	.16	.86	.00
6.0000	6.0000	1.10	38.33	50.79	1.28	2.73	.63	.47	.23	.85	.00
6.0000	6.0000	.90	49.03	75.88	.65	1.25	.50	.52	.40	.75	.00
6.0000	6.0000	.90	49.03	75.88	1.03	1.80	.70	.57	.39	.72	.00
6.0000	6.0000	.90	49.03	75.88	1.23	2.40	.90	.51	.38	.77	.00
6.0000	6.0000	.90	49.03	75.88	1.68	3.20	.70	.53	.22	.82	.00
6.0000	6.0000	.70	65.31	125.43	.58	1.25	.45	.46	.36	.81	.00
6.0000	6.0000	.70	65.31	125.43	1.00	2.18	.88	.46	.40	.79	.00
6.0000	6.0000	.70	65.31	125.43	1.33	2.60	1.00	.51	.38	.77	.00
6.0000	6.0000	.70	65.31	125.43	1.68	3.08	.93	.55	.30	.78	.00
6.0000	6.0000	.55	84.81	203.17	.68	1.35	.40	.50	.30	.81	.00
6.0000	6.0000	.55	84.81	203.17	1.11	1.83	.58	.61	.32	.73	.00
6.0000	6.0000	.55	84.81	203.17	1.35	2.45	.65	.55	.27	.79	.00
6.0000	6.0000	.55	84.81	203.17	1.55	2.70	.80	.57	.30	.76	.00
8.0000	8.0000	1.10	42.20	50.79	.68	1.33	.43	.51	.32	.80	.00
8.0000	8.0000	1.10	42.20	50.79	1.28	2.60	.40	.49	.15	.86	.00
8.0000	8.0000	1.10	42.20	50.79	1.75	3.78	.58	.46	.15	.87	.00
8.0000	8.0000	1.10	42.20	50.79	1.83	3.80	.60	.48	.16	.86	.00
8.0000	8.0000	.90	54.91	75.88	.50	.93	.33	.54	.35	.76	.00
8.0000	8.0000	.90	54.91	75.88	1.00	1.95	.70	.51	.36	.78	.00
8.0000	8.0000	.90	54.91	75.88	1.70	2.75	.70	.62	.25	.74	.00
8.0000	8.0000	.90	54.91	75.88	1.93	3.38	.88	.57	.26	.78	.00
8.0000	8.0000	.70	74.08	125.43	.40	1.03	.33	.39	.32	.86	.00
8.0000	8.0000	.70	74.08	125.43	.95	1.33	.33	.72	.25	.65	.00
8.0000	8.0000	.70	74.08	125.43	1.63	2.73	.98	.60	.36	.72	.00
8.0000	8.0000	.70	74.08	125.43	1.90	3.30	.85	.58	.26	.78	.00
8.0000	8.0000	.55	96.88	203.17	.30	.65	.30	.46	.46	.76	.00
8.0000	8.0000	.55	96.88	203.17	.68	1.28	.53	.53	.41	.74	.00
8.0000	8.0000	.55	96.88	203.17	1.03	2.05	.85	.50	.41	.76	.00
8.0000	8.0000	.55	96.88	203.17	1.30	2.53	.83	.51	.33	.79	.00
6.0000	4.0000	1.10	32.77	50.79	.00	1.28	.68	.00	.53	.85	2.00
6.0000	4.0000	1.10	32.77	50.79	.15	2.08	1.13	.07	.54	.84	2.00
6.0000	4.0000	1.10	32.77	50.79	.23	2.20	1.20	.10	.55	.83	2.00
6.0000	4.0000	1.10	32.77	50.79	.13	1.95	1.05	.07	.54	.84	2.00
6.0000	4.0000	.90	41.25	75.88	.00	1.20	.70	.00	.58	.81	2.00
6.0000	4.0000	.90	41.25	75.88	.28	2.00	1.10	.14	.55	.82	2.00
6.0000	4.0000	.90	41.25	75.88	.48	2.55	1.45	.19	.57	.80	2.00

Solid Breakwater-Regular Waves

Hs (in.)	h (in.)	f (Hz.)	L (in.)	LO (in.)	Ht (in.)	Hi (in.)	Hr (in.)	Kt	Kr	Kd	F (in.)
6.0000	4.0000	.90	41.25	75.88	.20	2.50	1.20	.08	.48	.87	2.00
6.0000	4.0000	.70	54.27	125.43	.00	1.05	.65	.00	.62	.79	2.00
6.0000	4.0000	.70	54.27	125.43	.20	1.70	1.10	.12	.65	.75	2.00
6.0000	4.0000	.70	54.27	125.43	.33	2.25	1.15	.15	.51	.85	2.00
6.0000	4.0000	.70	54.27	125.43	.56	2.35	1.15	.24	.49	.84	2.00
6.0000	4.0000	.55	69.98	203.17	.00	.75	.35	.00	.47	.88	2.00
6.0000	4.0000	.55	69.98	203.17	.00	.90	.40	.00	.44	.90	2.00
6.0000	4.0000	.55	69.98	203.17	.15	1.35	.45	.11	.33	.94	2.00
6.0000	4.0000	.55	69.98	203.17	.43	2.38	.88	.18	.37	.91	2.00
8.0000	6.0000	1.10	38.33	50.79	.25	1.73	1.03	.14	.59	.79	2.00
8.0000	6.0000	1.10	38.33	50.79	.83	2.43	1.33	.34	.55	.76	2.00
8.0000	6.0000	1.10	38.33	50.79	1.18	3.03	1.03	.39	.34	.86	2.00
8.0000	6.0000	1.10	38.33	50.79	1.13	3.40	1.00	.33	.29	.90	2.00
8.0000	6.0000	.90	49.03	75.88	.00	1.68	1.13	.00	.67	.74	2.00
8.0000	6.0000	.90	49.03	75.88	.53	2.20	1.50	.24	.68	.69	2.00
8.0000	6.0000	.90	49.03	75.88	.93	2.90	1.70	.32	.59	.74	2.00
8.0000	6.0000	.90	49.03	75.88	1.18	2.98	1.73	.40	.58	.71	2.00
8.0000	6.0000	.70	65.31	125.43	.00	1.60	.80	.00	.50	.87	2.00
8.0000	6.0000	.70	65.31	125.43	.40	2.35	1.25	.17	.53	.83	2.00
8.0000	6.0000	.70	65.31	125.43	.78	2.80	1.40	.28	.50	.82	2.00
8.0000	6.0000	.70	65.31	125.43	1.13	3.35	1.25	.34	.37	.86	2.00
8.0000	6.0000	.55	84.81	203.17	.00	1.08	.78	.00	.72	.69	2.00
8.0000	6.0000	.55	84.81	203.17	.40	2.05	.75	.20	.37	.91	2.00
8.0000	6.0000	.55	84.81	203.17	.70	2.73	.98	.26	.36	.90	2.00
8.0000	6.0000	.55	84.81	203.17	.85	2.90	.90	.29	.31	.90	2.00
4.0000	6.0000	1.10	38.33	50.79	1.21	1.43		.85			-2.00
4.0000	6.0000	1.10	38.33	50.79	1.60	2.15		.75			-2.00
4.0000	6.0000	1.10	38.33	50.79	1.78	1.92		.93			-2.00
4.0000	6.0000	1.10	38.33	50.79	1.58	1.66		.95			-2.00
4.0000	6.0000	.90	49.03	75.88	1.03	1.09		.94			-2.00
4.0000	6.0000	.90	49.03	75.88	1.40	1.76		.80			-2.00
4.0000	6.0000	.90	49.03	75.88	1.53	2.15		.71			-2.00
4.0000	6.0000	.90	49.03	75.88	1.66	1.79		.93			-2.00
4.0000	6.0000	.70	65.31	125.43	.83	1.11		.75			-2.00
4.0000	6.0000	.70	65.31	125.43	1.50	1.63		.92			-2.00
4.0000	6.0000	.70	65.31	125.43	1.80	2.05		.88			-2.00
4.0000	6.0000	.70	65.31	125.43	2.10	1.86		1.00			-2.00
4.0000	6.0000	.55	84.81	203.17	1.00	.98		1.00			-2.00
4.0000	6.0000	.55	84.81	203.17	1.53	1.63		.94			-2.00
4.0000	6.0000	.55	84.81	203.17	1.83	2.28		.80			-2.00
4.0000	6.0000	.55	84.81	203.17	2.03	1.95		1.00			-2.00
6.0000	8.0000	1.10	42.20	50.79	1.08	1.28	.08	.85	.06	.53	-2.00
6.0000	8.0000	1.10	42.20	50.79	1.45	2.60	.10	.56	.04	.83	-2.00
6.0000	8.0000	1.10	42.20	50.79	1.70	3.48	.13	.49	.04	.87	-2.00
6.0000	8.0000	1.10	42.20	50.79	1.75	4.08	.18	.43	.04	.90	-2.00
6.0000	8.0000	.90	54.91	75.88	.90	1.10	.10	.82	.09	.57	-2.00
6.0000	8.0000	.90	54.91	75.88	1.25	2.03	.18	.62	.09	.78	-2.00
6.0000	8.0000	.90	54.91	75.88	1.93	2.90	.30	.67	.10	.74	-2.00
6.0000	8.0000	.90	54.91	75.88	2.15	3.38	.38	.64	.11	.76	-2.00
6.0000	8.0000	.70	74.08	125.43	.98	.95	.30	1.00	.32		-2.00
6.0000	8.0000	.70	74.08	125.43	1.60	1.88	.58	.85	.31	.42	-2.00
6.0000	8.0000	.70	74.08	125.43	2.03	2.85	.95	.71	.33	.62	-2.00
6.0000	8.0000	.70	74.08	125.43	2.30	3.60	1.00	.64	.28	.72	-2.00
6.0000	8.0000	.55	96.88	203.17	.60	.78	.33	.77	.42	.47	-2.00
6.0000	8.0000	.55	96.88	203.17	1.33	1.70	.70	.78	.41	.47	-2.00
6.0000	8.0000	.55	96.88	203.17	1.80	2.53	.73	.71	.29	.64	-2.00
6.0000	8.0000	.55	96.88	203.17	1.98	2.88	.78	.69	.27	.67	-2.00

APPENDIX B

RUBBLE BREAKWATER--REGULAR WAVE DATA

Rubble Breakwater-Regular Waves

hs (in.)	h (in.)	f (Hz.)	L (in.)	L0 (in.)	Ht (in.)	Hi (in.)	Hr (in.)	Kt	Kr	F (in.)	Bn
4.000	6.000	1.100	38.330	50.8	1.930	3.125	.175	.62	.06	-2	12.000
4.000	6.000	1.100	38.330	50.8	1.780	3.525	.225	.50	.06	-2	12.000
4.000	6.000	.900	49.030	75.9	2.000	3.150	.350	.63	.11	-2	12.000
6.000	8.000	1.100	42.400	50.8	2.380	3.975	.275	.60	.07	-2	22.500
6.000	8.000	1.100	42.400	50.8	2.380	3.550	.150	.67	.04	-2	22.500
4.000	6.000	.700	65.310	125.4	2.350	2.800	.700	.84	.25	-2	12.000
4.000	6.000	1.100	38.330	50.8	1.700	2.475	.175	.69	.07	-2	12.000
4.000	6.000	.900	49.030	75.9	1.780	2.550	.350	.70	.14	-2	12.000
4.000	6.000	.700	65.310	125.4	1.880	2.450	.550	.77	.22	-2	12.000
6.000	8.000	.900	54.910	75.9	2.330	3.225	.375	.72	.12	-2	22.500
6.000	8.000	.700	74.080	125.4	2.800	3.400	.350	.82	.10	-2	22.500
4.000	6.000	.550	84.810	203.2	1.850	2.225	.475	.83	.21	-2	12.000
6.000	8.000	.900	54.910	75.9	2.150	2.925	.325	.74	.11	-2	22.500
4.000	6.000	.900	49.030	75.9	1.500	1.975	.275	.76	.14	-2	12.000
6.000	8.000	.550	96.880	203.2	2.500	2.950	.550	.85	.19	-2	22.500
4.000	6.000	.700	65.310	125.4	1.400	1.900	.450	.74	.24	-2	12.000
6.000	8.000	1.100	42.400	50.8	1.830	2.400	.150	.76	.06	-2	22.500
6.000	8.000	.700	74.080	125.4	2.350	2.675	.375	.88	.14	-2	22.500
4.000	6.000	.550	84.810	203.2	1.630	1.850	.450	.88	.24	-2	12.000
4.000	6.000	1.100	38.330	50.8	1.330	1.575	.125	.84	.08	-2	12.000
6.000	8.000	.550	96.880	203.2	2.080	2.500	.650	.83	.26	-2	22.500
4.000	6.000	.550	84.810	203.2	1.230	1.450	.350	.85	.24	-2	12.000
6.000	8.000	.900	54.910	75.9	1.780	1.950	.300	.91	.15	-2	22.500
4.000	6.000	.900	49.030	75.9	.930	1.275	.175	.73	.14	-2	12.000
4.000	6.000	.700	65.310	125.4	.880	1.200	.250	.73	.21	-2	12.000
6.000	8.000	.700	74.080	125.4	1.550	1.725	.275	.90	.16	-2	22.500
6.000	8.000	1.100	42.400	50.8	1.230	1.250	.150	.98	.12	-2	22.500
4.000	6.000	.550	84.810	203.2	.750	.975	.225	.77	.23	-2	12.000
6.000	8.000	.550	96.880	203.2	1.330	1.425	.325	.93	.23	-2	22.500
6.000	8.000	.900	54.910	75.9	.780	1.000	.150	.78	.15	-2	22.500
6.000	8.000	.700	74.080	125.4	.740	.925	.175	.80	.19	-2	22.500
6.000	8.000	.550	96.880	203.2	.640	.800	.250	.80	.31	-2	22.500
8.000	8.000	1.100	42.400	50.8	1.730	4.025	.225	.43	.06	0	36.000
6.000	6.000	1.100	38.330	50.8	1.300	3.025	.375	.43	.12	0	22.500
6.000	6.000	1.100	38.330	50.8	1.480	3.025	.275	.49	.09	0	22.500
4.000	4.000	.900	41.250	75.9	1.180	2.325	.275	.51	.12	0	12.000
4.000	4.000	1.100	32.770	50.8	1.080	2.150	.250	.50	.12	0	12.000
4.000	4.000	1.100	32.770	50.8	.950	2.150	.300	.44	.14	0	12.000
6.000	6.000	.900	49.030	75.9	1.700	3.200	.300	.53	.09	0	22.500
8.000	8.000	1.100	42.400	50.8	1.650	3.425	.175	.48	.05	0	36.000
4.000	4.000	.550	69.980	203.2	1.130	2.325	.425	.49	.18	0	12.000
4.000	4.000	.700	54.270	125.4	1.150	2.200	.300	.52	.14	0	12.000
6.000	6.000	.700	65.310	125.4	1.830	3.125	.575	.59	.18	0	22.500
4.000	4.000	.700	54.270	125.4	1.280	2.050	.300	.62	.15	0	12.000
4.000	4.000	1.100	32.770	50.8	1.000	1.825	.175	.55	.10	0	12.000
8.000	8.000	.900	54.910	75.9	1.850	3.275	.275	.56	.08	0	36.000
6.000	6.000	.900	49.030	75.9	1.450	2.600	.300	.56	.12	0	22.500
6.000	6.000	1.100	38.330	50.8	1.130	2.325	.125	.49	.05	0	22.500
4.000	4.000	.900	41.250	75.9	1.050	1.800	.200	.58	.11	0	12.000
6.000	6.000	.550	84.810	203.2	1.480	2.600	.450	.57	.17	0	22.500
6.000	6.000	.700	65.310	125.4	1.500	2.500	.400	.60	.16	0	22.500
8.000	8.000	.700	74.080	125.4	2.080	3.150	.450	.66	.14	0	36.000
8.000	8.000	.900	54.910	75.9	1.530	2.750	.350	.56	.13	0	36.000
4.000	4.000	.900	41.250	75.9	.830	1.550	.150	.54	.10	0	12.000
4.000	4.000	.550	69.980	203.2	1.000	1.575	.225	.63	.14	0	12.000

Rubble Breakwater-Regular Waves

hs	h	f	L	LO	Ht	Hi	Hr	Kt	Kr	F	Bn
8.000	8.000	1.100	42.400	50.8	1.150	2.300	.200	.50	.09	0	36.000
6.000	6.000	.900	49.030	75.9	1.100	1.975	.275	.56	.14	0	22.500
4.000	4.000	.700	54.270	125.4	1.030	1.475	.175	.70	.12	0	12.000
6.000	6.000	.550	84.810	203.2	1.150	2.150	.550	.53	.26	0	22.500
8.000	8.000	.700	74.080	125.4	1.700	2.600	.450	.65	.17	0	36.000
6.000	6.000	.700	65.310	125.4	1.330	1.925	.375	.69	.19	0	22.500
4.000	4.000	1.100	32.770	50.8	.710	1.200	.100	.59	.08	0	12.000
8.000	8.000	.550	96.880	203.2	1.530	2.475	.525	.62	.21	0	36.000
6.000	6.000	1.100	38.330	50.8	.800	1.500	.100	.53	.07	0	22.500
4.000	4.000	.550	69.980	203.2	.780	1.150	.200	.68	.17	0	12.000
6.000	6.000	.550	84.810	203.2	1.080	1.675	.425	.64	.25	0	22.500
8.000	8.000	.900	54.910	75.9	.950	1.750	.200	.54	.11	0	36.000
8.000	8.000	.550	96.880	203.2	1.380	2.025	.425	.68	.21	0	36.000
4.000	4.000	.900	41.250	75.9	.680	1.000	.100	.68	.10	0	12.000
4.000	4.000	.700	54.270	125.4	.750	.950	.100	.79	.11	0	12.000
8.000	8.000	.700	74.080	125.4	1.000	1.750	.350	.57	.20	0	36.000
6.000	6.000	.900	49.030	75.9	.750	1.250	.150	.60	.12	0	22.500
6.000	6.000	.700	65.310	125.4	.780	1.250	.250	.62	.20	0	22.500
8.000	8.000	1.100	42.400	50.8	.680	1.250	.100	.54	.08	0	36.000
6.000	6.000	.550	84.810	203.2	.680	1.100	.300	.62	.27	0	22.500
8.000	8.000	.550	96.880	203.2	.790	1.350	.300	.59	.22	0	36.000
4.000	4.000	.550	69.980	203.2	.450	.700	.150	.64	.21	0	12.000
8.000	8.000	.900	54.910	75.9	.450	.900	.100	.50	.11	0	36.000
8.000	8.000	.700	74.080	125.4	.490	.850	.150	.58	.18	0	36.000
8.000	8.000	.550	96.880	203.2	.370	.750	.150	.49	.20	0	36.000
8.000	6.000	1.100	38.330	50.8	.450	3.350	.350	.13	.10	2	36.000
8.000	6.000	1.100	38.330	50.8	.530	3.225	.375	.16	.12	2	36.000
8.000	6.000	.900	49.030	75.9	.480	3.025	.375	.16	.12	2	36.000
8.000	6.000	1.100	38.330	50.8	.430	2.425	.275	.18	.11	2	36.000
8.000	6.000	.700	65.310	125.4	.680	3.025	.675	.22	.22	2	36.000
6.000	4.000	1.100	32.770	50.8	.250	1.850	.350	.14	.19	2	22.500
6.000	4.000	.900	41.250	75.9	.370	2.050	.450	.18	.22	2	22.500
6.000	4.000	.900	41.250	75.9	.300	2.050	.450	.15	.22	2	22.500
6.000	4.000	1.100	32.770	50.8	.250	1.800	.300	.14	.17	2	22.500
6.000	4.000	.700	54.270	125.4	.400	2.175	.425	.18	.20	2	22.500
6.000	4.000	1.100	32.770	50.8	.280	1.800	.400	.16	.22	2	22.500
8.000	6.000	.900	49.030	75.9	.330	2.450	.300	.13	.12	2	36.000
6.000	4.000	.700	54.270	125.4	.430	2.125	.475	.20	.22	2	22.500
6.000	4.000	.900	41.250	75.9	.330	1.825	.325	.18	.18	2	22.500
8.000	6.000	.700	65.310	125.4	.470	2.450	.450	.19	.18	2	36.000
6.000	4.000	.550	69.980	203.2	.330	2.000	.500	.17	.25	2	22.500
8.000	6.000	.900	49.030	75.9	.250	2.075	.275	.12	.13	2	36.000
8.000	6.000	.550	84.810	203.2	.530	2.425	.525	.22	.22	2	36.000
6.000	4.000	.700	54.270	125.4	.340	1.575	.325	.22	.21	2	22.500
8.000	6.000	.700	65.310	125.4	.330	1.950	.550	.17	.28	2	36.000
8.000	6.000	1.100	38.330	50.8	.300	1.575	.225	.19	.14	2	36.000
8.000	6.000	.550	84.810	203.2	.580	2.050	.550	.28	.27	2	36.000
6.000	4.000	.550	69.980	203.2	.320	1.525	.275	.21	.18	2	22.500
6.000	4.000	1.100	32.770	50.8	.180	1.125	.225	.16	.20	2	22.500
6.000	4.000	.900	41.250	75.9	.210	1.150	.300	.18	.26	2	22.500
8.000	6.000	.550	84.810	203.2	.380	1.650	.450	.23	.27	2	36.000
8.000	6.000	.900	49.030	75.9	.190	1.275	.275	.15	.22	2	36.000
6.000	4.000	.700	54.270	125.4	.250	1.050	.250	.24	.24	2	22.500
6.000	4.000	.550	69.980	203.2	.280	1.125	.275	.25	.24	2	22.500
8.000	6.000	.700	65.310	125.4	.290	1.275	.375	.23	.29	2	36.000
8.000	6.000	.550	84.810	203.2	.280	.975	.325	.29	.33	2	36.000
6.000	4.000	.550	69.980	203.2	.200	.675	.175	.30	.26	2	22.500

APPENDIX C

RUBBLE BREAKWATER--IRREGULAR WAVE DATA

RUBBLE BREAKWATER-IRREGULAR WAVES

hs (in.)	h (in.)	f (Hz.)	L (in.)	LO (in.)	Ht (in.)	Hi (in.)	Hr (in.)	Kt	Kr	F (in.)
4.00	4.00	[illegible]	54.27	125.43	.73	1.32	.45	.55	.34	0
4.00	4.00	.70	54.27	125.43	.74	1.48	.51	.50	.34	0
4.00	4.00	.90	41.25	75.88	.56	1.15	.35	.49	.31	0
4.00	4.00	.90	41.25	75.88	.67	1.36	.43	.49	.31	0
6.00	6.00	.70	65.31	125.43	.53	1.06	.33	.50	.31	0
6.00	6.00	.70	65.31	125.43	.71	1.36	.41	.53	.31	0
6.00	6.00	.90	49.03	75.88	.67	1.62	.36	.42	.23	0
6.00	6.00	.90	49.03	75.88	.88	2.01	.46	.44	.23	0
8.00	8.00	.70	74.08	125.43	.81	1.39	.42	.59	.30	0
8.00	8.00	.70	74.08	125.43	1.27	2.15	.63	.59	.29	0
8.00	8.00	.90	54.91	75.88	.77	1.76	.42	.44	.24	0
8.00	8.00	.90	54.91	75.88	1.12	2.35	.54	.48	.23	0
6.00	4.00	.70	54.27	125.43	.30	1.27	.58	.24	.46	2
6.00	4.00	.70	54.27	125.43	.33	1.40	.61	.23	.43	2
6.00	4.00	.90	41.25	75.88	.27	1.32	.50	.20	.38	2
6.00	4.00	.90	41.25	75.88	.28	1.47	.57	.19	.39	2
8.00	6.00	.70	65.31	125.43	.33	1.02	.37	.32	.36	2
8.00	6.00	.70	65.31	125.43	.39	1.36	.48	.29	.35	2
8.00	6.00	.90	49.03	75.88	.35	1.68	.41	.21	.25	2
8.00	6.00	.90	49.03	75.88	.40	1.99	.49	.20	.25	2
4.00	6.00	.70	65.31	125.43	.90	.99	.29	.91	.29	-2
4.00	6.00	.70	65.31	125.43	1.18	1.32	.36	.89	.27	-2
4.00	6.00	.90	49.03	75.88	1.32	1.75	.37	.76	.21	-2
4.00	6.00	.90	49.03	75.88	1.54	2.07	.42	.75	.20	-2
6.00	8.00	.70	74.08	125.43	1.24	1.38	.33	.90	.24	-2
6.00	8.00	.70	74.08	125.43	1.75	2.12	.49	.83	.23	-2
6.00	8.00	.90	54.91	75.88	1.25	1.64	.28	.76	.17	-2
6.00	8.00	.90	54.91	75.88	1.60	2.38	.43	.67	.18	-2